THE ASPHALT HANDBOOK

ASPHALT INSTITUTE

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Foreword

This publication is the completely revised edition of The Asphalt Handbook. Over an approximately fifty year period the Handbook became the standard reference work in the field of asphalt technology and construction.

The Handbook was first issued in 1923 as the Asphalt Pocket Reference For Highway Engineers. This publication was the principal source of printed information of the newly formed Asphalt Institute. The Pocket Reference was reprinted at intervals without material change until 1937. In that year, owing to a rising demand for a more comprehensive volume incorporating design and construction principles in asphalt enginering, the newly designated Handbook was published.

Later editions during the next three decades broadened the scope of the Handbook and recognized continuing advances in asphalt engineering. Over this period there was a tremendous surge in asphalt technology. These advances complicated the task of subsequent Handbook revisions. During this period there were numerous editions and reprintings. In fact the 1962 edition had subsequent annual reprintings and the final 1965 edition was reprinted six times. Nearly 100,000 copies were distributed.

In 1974 distribution of the Handbook was suspended. This decision was brought about due to the increasing production costs as well as the technical difficulties of keeping the publication updated. The Institute had concurrently developed a complete set of other smaller, single subject manuals and informational publications. The just published Instructor's Guide (ES-1) was anticipated to take the place of the Handbook for a comprehensive textbook on all aspects of asphalt technology. While a success in its own right, ES-1 limited its audience and never satisfied the continual demand for reissuance of the Handbook.

After a nearly 15-year hiatus The Asphalt Handbook is back. Combining aspects of the original version and the updated ES-1, this Handbook is more comprehensive than any previous version. Containing over 600 pages with nearly 400 illustrations and tables, The Asphalt Handbook is the definitive informational source on asphalt technology.

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Chapter 1

Introduction to Asphalt

This chapter introduces the field of asphalt technology. It presents a broad overview of past and present uses of asphalt as well as an introduction to some of the basic concepts and definitions associated with asphalt.

SECTION 1.1 Historical Review
- ☐ Background
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- ☐ Summary

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1.1 Historic Review

Background

The word "asphalt" is believed to be derived from the ancient Accadian term "asphaltic." The term was adopted by the Homeric Greeks meaning "to make firm or stable." "Asphaltic" was carried over to late Latin, French "asphalte", and ultimately English "asphalt." From its ancient past to the present, asphalt has been used as a cement for bonding, coating and waterproofing objects. Asphalt truly is one of nature's most versatile products.

Asphalt is man's oldest engineering material, having been used since the dawn of civilization. In Sumeria, about 6000 B.C., there existed a thriving shipbuilding industry that produced and used asphalt. In building the ziggurats, of which the Tower of Babel was only one of many, asphalt was used for mortar. Near the cities of Sodom and Gomorrah, there was a thriving asphalt production. Asphalt was used by the Egyptians as a waterproofing material as early as 2600 B.C. and from the Persian word for asphalt, mumiya, we derive the word mummy. In the ancient world, asphalt continued to be used widely as a mortar for building and paving blocks, caulking for ships, and waterproofing in numerous applications.

These early asphalts occurred naturally. They were found in geologic strata both as soft, readily workable "mortars" and also as hard, friable black veins of rock formations. The soft asphalt material is typified in the Trinidad Lake depository, on the Island of Trinidad, in Bermudez Lake in Venezuela, and in extensive "tar sands" throughout western Canada. These "soft asphalt" products have come to be referred to as natural asphalts and were used extensively until the early part of this century.

In the early 1900's, the discovery of refining asphalt from crude petroleum and the overwhelming popularity of the automobile created an expanding industry. Asphalt appeared to be a cheap and inexhaustible resource that would be used for smooth, modern roads and numerous other applications (Figure 1.1).

As the asphalt paving industry blossomed, the physical properties and character of asphalt needed to be determined. To ensure asphalt roadways were durable and reliable, numerous tests and procedures were developed in the early 1900's. Probably the first scientific approach to designing asphalt paving mixes was determining an asphalt content based on aggregate voids.

In order for design samples to represent field performance, compacted samples needed to contain desired void and asphalt cement contents. In the middle 1920's, one of the earliest methods of evaluating the physical properties of a compacted asphalt mixture was developed by Prevost Hubbard and F.C. Field of the Asphalt Institute. The empirical Hubbard-Field test indicated the stability of a mix by a punching-shear type of failure.

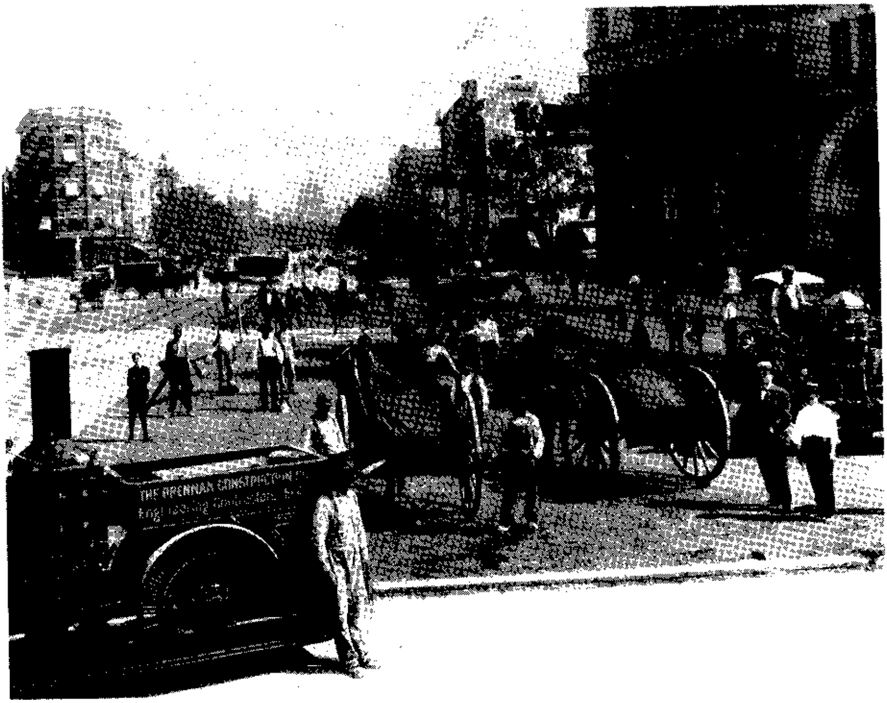


Figure 1.1 **Pavement Placement—1907, on Pennsylvania Avenue, Washington, D.C.**

Francis Hveem developed another test in the 1930's to ensure compacted asphalt pavement mixtures were stable and did not bleed excessive asphalt. The principles of the Hveem method are based on triaxial compression and an estimate of asphalt content determined by the surface area of the aggregate. The Hveem Method is still practiced today.

With the advent of World War II, a simple test was required for proper design and construction of the military airfields. An asphalt mix design method was conceived by Bruce Marshall to determine the desired density and asphalt content to withstand the heavy aircraft wheel loads. The Marshall method provided satisfactory design criteria. Today, this empirical method is extensively used throughout the world.

Historical Milestones

Prehistoric: Skeletons of prehistoric animals are preserved intact to present day in asphalt surface deposits, La Brea Pit, Los Angeles, California.

3000 B.C. - 476: Following discovery of the wheel, the first man-made roads were built in Southwestern Asia. Later, the Persians developed roads, in many instances with the use of asphalt. The Romans effected early colonization by means of a military road system spanning their empire.

1599: First classification of asphalt and attempts to trace connection between asphalt and petroleum.

1777: First exposition of the modern theory of the origin of asphalt by P.C. LeSage, France.

1780: First asphalt composition of prepared roofing, Sweden.

1802: Rock asphalt used in France for floor, bridge and sidewalk surfacing.

1815: In London, John L. McAdam began building and improving roads with an interlocked and compacted stone surface. This "macadam" innovation had a profound impact on the roadways of the future.

1824: First use of asphalt paving blocks.

1829: First use of asphalt-mastic pavements, Lyons, France.

1837: The first account of asphalt deposits in the United States revealed in Connecticut. Also, publication of first exhaustive treatise on the chemistry of asphalt, by J.B. Borissingault in Paris.

1838: Rock asphalt imported and used in sidewalk construction in Philadelphia.

1844: First composition asphalt roofing in the United States.

1858: First modern asphalt road laid in Paris, composed of 50 mm (2 inches) of compressed asphalt mastic.

1869: First compacted asphalt pavement in London.

1870: First U.S. asphalt roadways laid in Newark, New Jersey.

1876: First sheet asphalt pavement laid on Pennsylvania Avenue in Washington, D.C., with imported asphalt from Trinidad Lake.

1899: Barber Asphalt Paving Co. heavily involved in early paving equipment, development and construction of nearly half of all the early sheet asphalt pavements.

1901: Warren Brothers built the first successful, reproducible asphalt concrete pavements. They also pioneered development of early hot-mix asphalt facilities.

1902: Approximately 18,000 tonnes (20,000 tons) of asphalt produced in the United States from the recent discovery of petroleum asphalt refining.

1910: First drum mixers and dryer-mixers were used at mixing facilities.

1919: The Asphalt Association was founded (later renamed the Asphalt Institute) in New York City to serve the users of petroleum asphalt through programs of engineering, research and education.

1921: The Federal-Aid Road Act of 1916 was amended, enabling the U.S. to progress in building and improving roadways.

1930: The Asphalt Institute was instrumental in the crusade to develop early pavement material and construction procedures.

1941: The U.S. Defense Act of 1941 authorized the building of roads essential to the war effort.

1946: After World War II, the U.S. started a new era of roadbuilding. In the next ten years 80,000 kilometers (50,000 miles) of new roads were constructed. In the same period, enough automobiles were manufactured to stretch 322,000 kilometers (200,000 miles) bumper-to-bumper.

1956: President Eisenhower signed the history making Federal Aid Highway Act, authorizing the building of 66,000 kilometers (41,000 miles) of the U.S. Interstate and Defense Highways System.

1960: Full-Depth pavement concept was conceived.

1979: Record high usage of asphalt in the U.S., exceeding 34 megatonnes (38 million tons).

1.2 Modern Asphalt Usage

Asphalt

Asphalt is of particular interest to the engineer because it is a strong cement, readily adhesive, highly waterproof, and durable. It is a plastic substance that imparts controllable flexibility to mixtures of mineral aggregates with which it is usually combined. It is, moreover, highly resistant to the action of most acids, alkalies, and salts. Although a solid or semi-solid at ordinary atmospheric temperatures, asphalt may be readily liquefied by applying heat, or by dissolving it in petroleum solvents of varying volatility, or by emulsifying it.

Modern asphalt is a natural constituent of petroleum. Most crude petroleums contain some asphalt, and sometimes crude oil may be almost entirely asphalt. There are some crude oils, however, that contain no asphalt. On the basis of their asphalt content, crude oils are often broadly described or classed as:

1. Asphalt base crude;
2. Paraffin base crude (containing paraffin but no asphalt); and
3. Mixed-base crude (containing both paraffin and asphalt).

Crude petroleum from oil wells is separated into its constituents or fractions in a refinery. Principally, this separation is accomplished by distillation. After separation, the constituents are further refined or processed into products meeting specific requirements. Hence, asphalt, paraffin, gasoline, lubricating oil, and other highly useful products are the output of an oil refinery, depending on the nature of the crude oil being processed.

Because asphalt is the base or heavy constituent of crude oil, it does not evaporate or boil off when crude oil is distilled. Accordingly, asphalt is obtained as a residue or residual product, and is valuable and essential for a great variety of engineering and architectural uses.

Practically all asphalt used in the United States is produced by modern oil refineries and is called *petroleum asphalt.*

Asphalt is also a *bituminous* material because it contains *bitumen,* which is a hydrocarbon material soluble in carbon disulfide (CS_2). Tar obtained from the destructive distillation of soft coal also contains bitumen. Consequently, both petroleum asphalt and coal tar are jointly referred to as bituminous materials. However, petroleum asphalt should not be confused with coal tar because their properties differ greatly. Petroleum asphalt is composed almost entirely of bitumen, whereas in coal tar the bitumen content is relatively low. In view of these differences, it is imperative that coal-tar products and petroleum asphalts be considered and treated as entirely separate entities.

Asphalt is used principally in this country for various paving purposes. However, considerable asphalt is consumed in the roofing industry as well as for numerous miscellaneous products. The next section discusses this in greater detail.

Petroleum asphalt for use in pavements is usually called paving asphalt or asphalt cement to distinguish it from asphalt made for non-paving uses, such as roofing and industrial purposes.

Paving asphalt at normal atmospheric (ambient) temperatures is a black, sticky, semisolid, highly viscous material. It is composed primarily of complex hydrocarbon molecules, but it also contains other atoms, such as oxygen, nitrogen, and sulfur. Because paving asphalt is sticky, it adheres to aggregate particles and can be used to cement or bind them into asphalt concrete. Paving asphalt is waterproof and is unaffected by most acids, alkalies, and salts. It is called a *thermoplastic* material because it softens as it is heated and hardens as it is cooled. This unique combination of characteristics and properties is a fundamental reason why asphalt is an important paving material.

Asphalt pavements are sometimes incorrectly called flexible pavements, perhaps as a result of asphalt being a viscous, thermoplastic material.

Petroleum asphalt, as previously stated, is the principal paving asphalt in use today, although natural or native asphalt is still used in the United States and other countries. The extent of native asphalt used, however, is relatively small. In fact, only a few thousand tons of natural asphalt is used in the United States each year. Natural asphalt is durable and has been used throughout history. Modern petroleum asphalt has the same durable qualities, but it has the important additional advantage of being refined to a uniform condition free from extraneous organic and mineral matter. Natural asphalt, however, is nonuniform and contains various amounts of extraneous material.

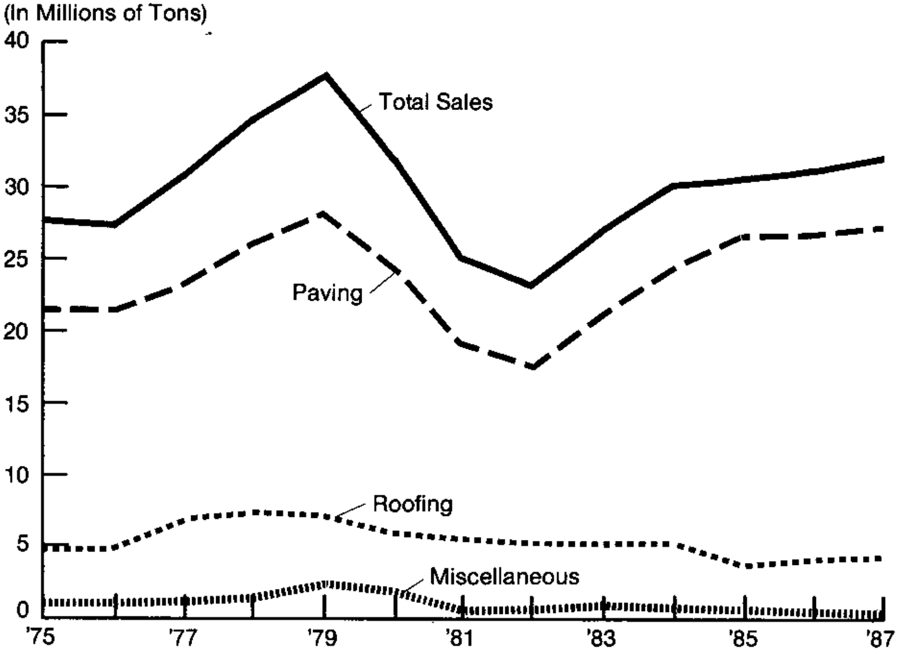
Uses of Asphalt

Asphalt is used for a great variety of purposes. Annual worldwide consumption of asphalt is over 90 megatonnes (100 million tons). The production and consumption of asphalt in the United States has shown a steady growth from about 1.4 megatonnes (1.5 million tons) in 1920 to about 34 megatonnes (38 million tons) in 1979 (Figure 1.2).

The use of asphalt in roadbuilding has grown concurrently with overall consumption. Of the approximately 3,466,000 kilometers (2,154,000 miles) of paved roads in the United States, 3,267,000 kilometers (2,030,000 miles), or 94% are surfaced with asphalt (Figure 1.3).

The major coverage of this manual will be to describe asphalt construction and design procedures as they relate to the predominant use of asphalt: pavements carrying vehicular traffic. However, as Table 1.1 indicates, asphalt is used today, as in the past, for a multitude of special applications and products.

Annual Sales of Petroleum Asphalt to Domestic Consumers in U.S.A.



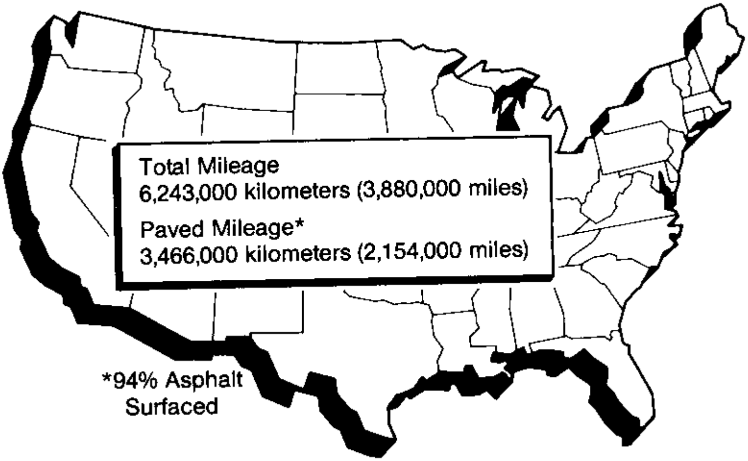
Source: U.S. Bureau of Mines
Department of Energy
Asphalt Institute Annual Survey

Charted by the Asphalt Institute

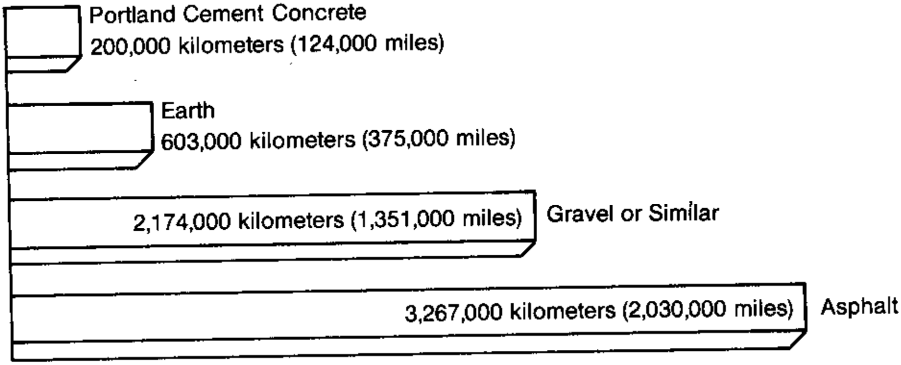
*1 ton = 0.907 tonnes

Figure 1.2 U.S. Asphalt Consumption.

Total Existing Length of Roads and Streets, U.S.A.


Total Mileage
6,243,000 kilometers (3,880,000 miles)
Paved Mileage*
3,466,000 kilometers (2,154,000 miles)
*94% Asphalt Surfaced

Total Existing Mileage of Roads and Streets U.S.A.



Source: Federal Highway Administration.

Figure 1.3 U.S. Road Mileage.

Table 1.1 Some Uses and Applications of Asphalt

AGRICULTURE
- (Also see Buildings, Hydraulics and Paving)
- Cattle Sprays
- Dampproofing and waterproofing buildings, structures
- Disinfectants
- Fence Post coating
- Mulches
- Mulching paper
- Paved barn floors, barnyards, feed platforms, etc.
- Protecting tanks, vats, etc.
- Protection for concrete structures
- Tree paints
- Water and moisture barriers (above & below ground)
- Wind and water erosion control
- Weather modification areas

BUILDINGS
- (Also see Industrial, Paving)

Floors
- Dampproofing and waterproofing
- Floor compositions, tiles, coverings
- Insulating fabrics, papers
- Step treads

Roofing
- Building papers
- Built-up roof adhesives, felts, primes
- Caulking compounds
- Cement waterproofing compounds
- Cleats for roofing
- Glass wool compositions
- Insulating fabrics, felts, papers
- Joint filler compounds
- Laminated roofing, shingles
- Liquid roof coatings
- Plastic cements
- Shingles

Walls, Siding, Ceilings
- Acoustical blocks, compositions, felts
- Architectural decoration
- Bricks
- Brick siding
- Building blocks, papers
- Dampproofing coatings, compositions
- Insulating board, fabrics, felts, paper
- Joint filler compounds
- Masonry coatings
- Plaster boards
- Putty, asphalt
- Siding compositions
- Soundproofing
- Stucco base
- Wallboard

Miscellaneous
- Air drying paints, varnishes
- Artificial lumber
- Ebonized lumber
- Insulating paints
- Plumbing, pipes
- Treated awnings

HYDRAULICS & EROSION CONTROL
- Canal linings, sealants
- Catchment areas, basins
- Dam groutings
- Dam linings, protection
- Dike protection
- Ditch linings
- Drainage gutters, structures
- Embankment protection
- Groins
- Jetties
- Levee protection
- Mattresses for levee and bank protection
- Membrane linings, waterproofing
- Ore leaching pads
- Reservoir linings
- Revetments
- Sand dune stabilization
- Sewage lagoons, oxidation ponds
- Swimming pools
- Waste ponds
- Water barriers

INDUSTRIAL
- Aluminum foil compositions using asphalt
- Backed felts
- Conduit insulation, lamination
- Insulating boards
- Paint compositions
- Papers
- Pipe wrapping
- Roofing, shingles

Automotive
- Acoustical compositions, felts
- Brake linings
- Clutch facings
- Floor sound deadeners
- Friction elements
- Insulating felts
- Panel boards
- Shim strips
- Tacking strips
- Underseal

Electrical
- Armature carbons, windings
- Battery boxes, carbons
- Electrical insulating compounds, papers, tapes, wire coatings
- Junction box compound
- Molded conduits

Table 1.1 Some Uses and Applications of Asphalt (Continued)

INDUSTRIAL (Cont.)

Compositions
- Black grease
- Buffing compounds
- Cable splicing compound
- Embalming
- Etching compositions
- Extenders, rubber, other
- Explosives
- Fire extinguisher compounds
- Joint fillers
- Lap cement
- Lubricating grease
- Pipe coatings, dips, joint seals
- Plastic cements
- Plasticizers
- Preservatives
- Printing inks
- Well drilling fluid
- Wooden cask liners

Impregnated, Treated Materials
- Armored bituminized fabrics
- Burlap impregnation
- Canvas treating
- Carpeting medium
- Deck cloth impregnation
- Fabrics, felts
- Mildew prevention
- Packing papers
- Pipes and pipe wrapping
- Planks
- Rugs, asphalt base
- Saw dust, cork, asphalt composition
- Textiles, waterproofing
- Tiles
- Treated leather
- Wrapping papers

Paints, Varnishes, etc.
- Acid-proof enamels, mastics, varnishes
- Acid-resistant coatings
- Air-drying paints, varnishes
- Anti-corrosive & anti-fouling paints
- Anti-oxidants and solvents
- Base for solvent compositions
- Baking and heat resistant enamels
- Boat deck sealing compound
- Lacquers, japans
- Marine enamels

Miscellaneous
- Belting
- Blasting fuses
- Briquette binders
- Burial vaults
- Casting molds
- Clay articles

Miscellaneous (Cont.)
- Clay pigeons
- Depilatory
- Expansion joints
- Flower pots
- Foundry cores
- Friction tape
- Gaskets
- Imitation leather
- Mirror backing
- Photograph records
- Rubber, molded compositions
- Shoe fillers, soles
- Table tops

PAVING
- (Also see Hydraulics, Agriculture, Railroad, Recreation)
- Airport runways, taxiways, aprons, etc.
- Asphalt blocks
- Brick fillers
- Bridge deck surfacing
- Crack fillers
- Curbs, gutters, drainage ditches
- Floors for buildings, warehouses, garages, etc.
- Highways, roads, streets, shoulders
- Parking lots, driveways
- PCC Underseal
- Roof-deck parking
- Sidewalk, footpaths
- Soil stabilization

RAILROAD
- Ballast-treatment
- Curve lubricant
- Dust laying
- Paved ballast, sub-ballast
- Paved crossings, freight yards, station platforms
- Rail fillers
- Railroad ties
- Tie impregnating, stabilization

RECREATION
- Paved surfaces for:
- Dance pavilions
- Drive-in movies
- Gymnasiums, sports arenas
- Playgrounds, school yards
- Race tracks
- Running tracks
- Skating rinks
- Swimming & wading pools
- Tennis courts, handball courts

Terms Relating to Asphalt and Its Uses

Asphalt Materials

Asphalt—A dark brown to black cementitious material in which the predominating constituents are bitumens which occur in nature or are obtained in petroleum processing. (ASTM* Designation D8.)

Asphalt is a constituent in varying proportions of most crude petroleums.

Asphalt Blocks—Asphalt concrete molded under high pressure. The type of aggregate mixture composition, amount and type of asphalt, and the size and thickness of the blocks are varied to suit usage requirements.

Asphalt, Blown or Oxidized—Asphalt that is treated by blowing air through it at elevated temperature to give it characteristics desired for certain special uses such as roofing, pipe coating, undersealing portland cement concrete pavements, membrane envelopes, and hydraulic applications.

Asphalt, Catalytically Blown—An air-blown asphalt produced by using a catalyst during the blowing process.

Asphalt Cement—Asphalt that is refined to meet specifications for paving, industrial, and special purposes. (See Specifications for Asphalt Cement, ASTM Designations D 946 and D 3381, or AASHTO† Designations M 20 and M 226). Its penetration is usually between 40 and 300. The term is often abbreviated AC.

Asphalt, Cutback—See *Cutback Asphalt.*

Asphalt Joint Sealer—An asphalt product used for sealing cracks and joints in pavements and other structures.

Asphalt Joint Sealers, Preformed—Premolded strips of asphalt mixed with fine mineral substances, fibrous materials, cork, sawdust, or similar materials; manufacturerd in dimensions suitable for construction joints.

Asphalt, Emulsified—See *Emulsified Asphalt.*

Asphalt, Mineral Filled—Asphalt containing finely divided mineral matter passing 75 μm (No. 200) sieve.

Asphalt, Natural (Native)—Asphalt occurring in nature which has been derived from petroleum by natural processes of evaporation of volatile fractions leaving the asphalt fractions. The native asphalts of most importance are found in the Trinidad and Bermudez Lake deposits. Asphalt from these sources often is called *Lake Asphalt.*

Asphalt Paint—A fluid asphalt product sometimes containing small amounts of other materials such as lampblack, aluminum flakes, and mineral pigments.

Asphalt Panels, Premolded—Generally made with a core of asphalt, minerals, and fibers, covered on each side by a layer of asphalt-impregnated felt or fabric, coated on the outside with hot applied asphalt. The panels are made under pressure and heat to a width of 0.9 to 1.2 m (3 to 4 ft) by 3 to 25 mm (⅛ to 1 in.) thick, and to any desired length.

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*American Society for Testing and Materials.

†American Association of State Highway and Transportation Officials.

Asphalt, Petroleum—Asphalt refined from crude petroleum.

Asphalt Planks—Premolded mixtures of asphalt, fiber and mineral filler, sometimes reinforced with steel or fiberglass mesh. They are usually made in 0.9 to 2.4 m (3 to 8 ft) lengths and 150 to 300 mm (6 to 12 in.) wide. Asphalt planks may also contain mineral grits which maintain a sandpaper texture throughout their life.

Asphalt, Powdered—Solid or hard asphalt crushed or ground to a fine state of subdivision.

Asphalt Primer—A cutback asphalt product of low viscosity that penetrates into an aggregate base/subbase upon application.

Asphalt, Rock—Porous rock such as sandstone or limestone that has become impregnated with natural asphalt through geologic process.

Asphalt, Semi-Solid—Asphalt that is intermediate in consistency between emulsified and cutback asphalt products and solid or hard asphalt—that is, normally has a penetration between 10 and 300.

Asphalt, Solid or Hard—Asphalt having a normal penetration of less than 10.

Bitumen—A mixture of hydrocarbons of natural or pyrogenous origin, or a combination of both; frequently accompanied by nonmetallic derivatives which may be gaseous, liquid, semisolid, or solid; and which are completely soluble in carbon disulfide.

Cutback Asphalt—Asphalt cement which has been liquefied by blending with petroleum solvents (also called diluents). Upon exposure to atmospheric conditions the diluents evaporate, leaving the asphalt cement to perform its function of cementing and waterproofing.

a. Rapid-Curing (RC) Asphalt—Cutback asphalt composed of asphalt cement and a naphtha or gasoline-type diluent of high volatility. (See Specifications for Rapid-Curing Asphalt, ASTM Designation D 2028 or AASHTO Designation M 81).

b. Medium-Curing (MC) Asphalt—Cutback asphalt composed of asphalt cement and kerosene-type diluent of medium volatility. (See Specifications for Medium-Curing Asphalt, ASTM Designation D 2027 or AASHTO Designation M 82).

c. Slow-Curing (SC) Asphalt—Cutback asphalt composed of asphalt cement and oils of low volatility. (See Specifications for Slow-Curing Asphalt, ASTM Designation D 2026).

d. Road-Oil—A heavy petroleum oil, usually one of the Slow-Curing (SC) grades. (See Specifications for Slow-Curing Asphalt, ASTM Designation D 2026).

Emulsified Asphalt—An emulsion of asphalt cement and water that contains a small amount of an emulsifying agent, a heterogeneous system containing two normally immiscible phases (asphalt and water) in which the water forms the continuous phase of the emulsion, and minute globules of asphalt form the discontinuous phase. Emulsified asphalts may be of either the anionic, electronegatively charged asphalt globules, or cationic, electropositively charged asphalt globule types, depending upon the emulsifying agent. (See Specifications for Emulsified Asphalts, ASTM Designation D 977 or AASHTO Designation M 140 and Specifications for Cationic Emulsified Asphalts, ASTM Designation D 2397 or AASHTO Designation M 208).

Flux or Flux Oil—A thick, relatively nonvolatile fraction of petroleum which may be used to soften asphalt to a desired consistency; often used as base stock for manufacture of roofing asphalts.

Gilsonite—A form of natural asphalt, hard and brittle, occurring in rock crevices or veins from which it is mined.

Asphalt Pavements and Surface Treatments

Aggregate—Any hard, inert, mineral material used for mixing in graduated fragments. It includes sand, gravel, crushed stone, and slag.

Aggregate, Coarse—That retained on the 2.36 mm (No. 8) sieve.

Aggregate Coarse-Graded—One having a continuous grading in sizes of particles from coarse through fine with a predominance of coarse sizes.

Aggregate, Fine—That passing the 2.36 mm (No. 8) sieve.

Aggregate, Fine-Graded—One having a continuous grading in sizes of particles from coarse through fine with a predominance of fine sizes.

Aggregate, Macadam—A coarse aggregate of uniform size usually of crushed stone, slag, or gravel.

Aggregate, Open-Graded—One containing little or no fine aggregate and mineral filler or in which the void spaces in the compacted aggregate are relatively large.

Aggregate, Well-Graded—Aggregate that is uniformly graded from the maximum size down to filler with the object of obtaining an asphalt mix with a controlled void content and high stability.

Asphalt Base Course—A foundation course or pavement layer consisting of mineral aggregate, bound together with asphalt material on which successive course(s) are placed.

Asphalt Block Pavements—Pavements in which the surface course is constructed of asphalt/aggregate blocks. These blocks are laid in regular courses as in the case of brick pavements.

Asphalt Concrete—High quality, thoroughly controlled hot mixture of asphalt cement and well-graded, high quality aggregate, thoroughly compacted into a uniform dense mass typified by ASTM Specification D 3515.

Asphalt Emulsion Slurry Seal—A mixture of slow-setting emulsified asphalt, fine aggregate and mineral filler, with water added to produce slurry consistency.

Asphalt Fog Seal—A light application of a slow-setting asphalt emulsion diluted with water and without mineral aggregate cover.

Asphalt Intermediate Course (sometimes called Binder Course)—An asphalt pavement course between a base course and an asphalt surface course.

Asphalt Leveling Course—A course (asphalt aggregate mixture) of variable thickness used to eliminate irregularities in the contour of an existing surface prior to superimposed treatment or construction.

Asphalt Macadam—A type of pavement construction using a coarse, open-graded aggregate that is usually produced by crushing and screening stone, slag or gravel. Such aggregate is called macadam aggregate. Asphalt may be incorporated in macadam construction either by penetration or by mixing.

Asphalt Mastic—A mixture of asphalt and fine mineral material in such proportions that it may be poured hot or cold into place and compacted by troweling to a smooth surface.

Asphalt Overlay—One or more courses of asphalt construction on an existing pavement. The overlay may include a leveling course to correct the contour of the old pavement, followed by uniform course or courses to provide needed thickness.

Asphalt Pavements—Pavements consisting of a surface course of mineral aggregate coated and cemented together with asphalt cement on supporting courses such as asphalt bases; crushed stone, slag, or gravel; or on portland cement concrete, brick, or block pavement.

Asphalt Pavement Structure—A pavement structure with all its courses of asphalt-aggregate mixtures, or a combination of asphalt courses and untreated aggregate courses placed above the subgrade or improved subgrade.

Asphalt Prime Coat—An application of a low viscosity cutback asphalt product to an absorbent surface. It is used to prepare an untreated base for an asphalt surface. The prime penetrates into the base and plugs the voids, hardens the top and helps bind it to the overlying asphalt course.

Asphalt, Sand—A mixture of sand and asphalt cement or cutback or emulsified asphalt. It may be prepared with or without special control of aggregate grading and may or may not contain mineral filler. Either mixed-in-place or plant-mix construction may be employed. Sand-asphalt is used in construction of both base and surface courses.

Asphalt Seal Coat—A thin asphalt surface treatment used to waterproof and improve the texture of an asphalt wearing surface. Depending on the purpose, seal coats may or may not be covered with aggregate. The main types of seal coats are aggregate seals, fog seals, emulsion slurry seals and sand seals.

Asphalt, Sheet—A hot mixture of asphalt cement with clean angular, graded sand and mineral filler. Its use ordinarily is confined to surface course, usually laid on an intermediate or leveling course.

Asphalt Soil Stabilization (Soil Treatment)—Treatment of naturally-occurring non-plastic or moderately plastic soil with cutback or emulsified asphalt at ambient temperatures. Aeration and compaction of the asphalt-soil mixture produces water-resistant base and subbase courses of improved load-bearing qualities.

Asphalt Surface Course—The top course of an asphalt pavement, sometimes called asphalt wearing course.

Asphalt Surface Treatments—Asphalt surface treatment is a broad term embracing several types of asphalt or asphalt-aggregate applications. usually less than 25 mm (1 inch) thick, to a road surface. The types range from a light application of emulsified or cutback asphalt to single or multiple surface layers made up of alternating applications of asphalt and aggregate.

Asphalt Tack Coat—A very light application of asphalt, usually asphalt emulsion diluted with water. It is used to ensure a bond between the surface being paved and the overlying course.

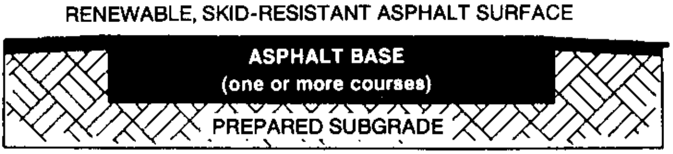
Base Course—The layer of material immediately beneath the surface or intermediate course. It may be composed of crushed stone, crushed slag, crushed or uncrushed gravel and sand, or combinations of these materials. It also may be bound with asphalt (see *Asphalt Base Course).*

Cold-Laid Plant Mixture—Plant mixes that may be spread and compacted at atmospheric temperature.

Thick-Lift Asphalt Construction—A construction practice in which the asphalt course is placed in one or more lifts of 100 mm (4 in.) or more compacted thickness.

Deep-Strength® Asphalt Pavement—The term *Deep-Strength* (registered by the Asphalt Institute with the U.S. Patent Office) certifies that the pavement is constructed of asphalt with an asphalt surface on an asphalt base and in accordance with design concepts established by the Institute. (See latest edition of *Thickness Design* manual, MS-1).

Full-Depth® Asphalt Pavement—The term *Full-Depth* (registered by the Asphalt Institute with the U.S. Patent Office) certifies that the pavement is one in which asphalt mixtures are employed for *all* courses above the subgrade or improved subgrade. A *Full-Depth* asphalt pavement is laid directly on the prepared subgrade. (The mathematical symbol T_A denotes Full-Depth or Total Asphalt).



Hot-Mix Asphalt (HMA)—Asphalt/aggregate mixture produced at a batch or drum-mixing facility that must be spread and compacted while at an elevated temperature. To dry the aggregate and obtain sufficient fluidity of the asphalt cement, both must be heated prior to mixing—giving origin to the term "hot mix."

Mineral Dust—The portion of the fine aggregate passing the 75 μm (No. 200) sieve.

Mineral Filler—A finely divided mineral product at least 70 percent of which will pass a 75 μm (No. 200) sieve. Pulverized limestone is the most commonly manufactured filler, although other stone dust, hydrated lime, portland cement, fly ash and certain natural deposits of finely divided mineral matter are also used.

Mixed-In-Place (Road-Mix)—An asphalt course produced by mixing mineral aggregate and cutback or emulsified asphalt at the road site by means of travel plants, motor graders, or special road-mixing equipment.

Multiple Surface Treatment—Two or more surface treatments using asphalt and aggregate placed one on the other. The aggregate maximum size of each successive treatment is usually one-half that of the previous one, and the total thickness is about the same as the nominal maximum size aggregate particles of the first course. Or, a multiple surface treatment may be a series of single treatments that produces a pavement course up to 38 mm (1½ in.) or more. A multiple surface treatment is a denser wearing and waterproofing course than a single surface treatment, and it adds *some* strength but is not normally assigned a structural coefficient.

Pavement Structure—All courses of selected material placed on the foundation or subgrade soil, other than any layers or courses constructed in grading operations.

Pavement Structure Combination- or Composite-Type—When the asphalt pavement is on old portland cement concrete pavement, a portland cement concrete base, or other rigid-type base or on a granular base, the pavement structure is referred to as a combination- or composite-type pavement structure.

Plant Mix—A mixture, produced in an asphalt mixing facility that consists of mineral aggregate uniformly coated with asphalt cement, emulsified asphalt or cutback asphalt.

Road-Mix—See *Mixed-In-Place.*

Rock Asphalt Pavements—Pavements constructed of rock asphalt, natural or processed and treated with asphalt or flux as may be required for construction.

Seal Coat—See *Asphalt Seal Coat.*

Single Surface Treatments—A single application of asphalt to any kind of road surface followed immediately by a single layer of aggregate of as uniform size as practicable. The thickness of the treatment is about the same as the nominal maximum size aggregate particles. A single surface treatment is used as a wearing and waterproofing course.

Subbase—The course in the asphalt pavement structure immediately below the base course is called the subbase course. If the subgrade soil is of adequate quality it may serve as the subbase.

Subgrade—The soil prepared to support a structure or a pavement system. It is the foundation for the pavement structure. The subgrade soil sometimes is called "basement soil" or "foundation soil."

Subgrade, Improved—Subgrade, improved as a working platform (1) by the incorporation of granular materials or stabilizers such as asphalt cement, emulsion or cutback, lime, or portland cement, prepared to support a structure or a pavement system, or (2) any course or courses of select or improved material placed on the subgrade soil below the pavement structure.

Surface Treatment—See *Asphalt Surface Treatment.*

1.3 Asphalt Pavements for Vehicular Traffic

Basic Concepts

Introduction

The basic idea in building a road or parking area for all-weather use by vehicles is to prepare a suitable subgrade or foundation, provide necessary drainage, and construct a pavement that will:

1. Have sufficient total thickness and internal strength to carry expected traffic loads,
2. Have adequate compaction to prevent the penetration or internal accumulation of moisture, and
3. Have a top surface that is smooth, skid resistant, and resistant to wear, distortion, and deterioration by weather and deicing chemicals.

The subgrade ultimately carries all traffic loads. Therefore, the structural function of a pavement is to support a wheel load on the pavement surface and transfer and spread that load to the subgrade without exceeding either the strength of the subgrade or the internal strength of the pavement itself (Figure 1.4).

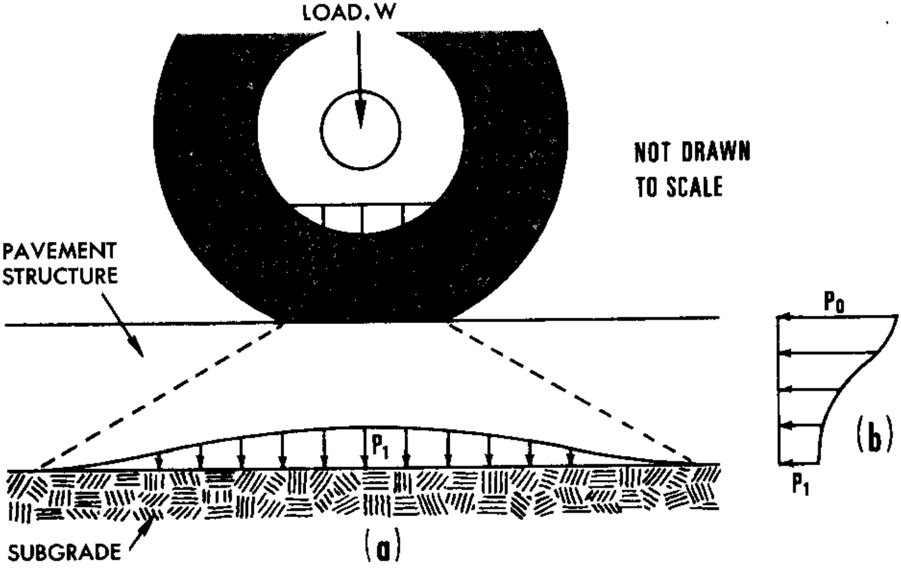


Figure 1.4 Spread of Wheel-Load Through Pavement Structure.

Figure 1.4 shows wheel load, W, being transmitted to the pavement surface through the tire at an approximately uniform vertical pressure, P_o. The pavement then spreads the wheel load to the subgrade so that the maximum pressure on the subgrade is only P_1. By proper selection of pavement materials and with adequate pavement thickness, P_1 will be small enough to be easily supported by the subgrade.

Descriptions and Definitions

Asphalt pavement is a general term applied to any pavement that has a surface constructed with asphalt (Figure 1.5). Normally, it consists of a surface course (layer) of mineral aggregate coated and cemented with asphalt; and one or more supporting courses, which may be:

1. Asphalt base, consisting of asphalt-aggregate mixtures;
2. Crushed stone (rock), slag, or gravel; and
3. Portland cement concrete.

An asphalt pavement structure consists of all courses or layers above the prepared subgrade or foundation. The upper or top layer is the asphalt wearing surface. It may range from less than 25 mm (1 in.) to more than 75 mm (3 in.) in thickness, depending on a variety of factors and circumstances explained in detail in the chapters on design, construction, and maintenance. and maintenance.

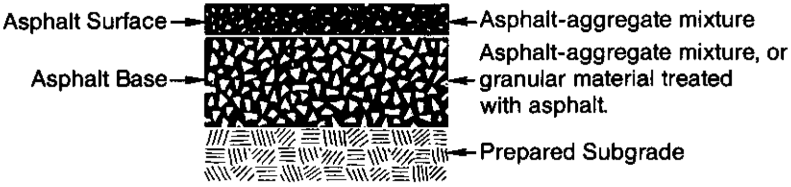
While a variety of bases and subbases may be used in asphalt pavement structures, often they consist of compacted granular materials (such as crushed rock, slag, gravel, sand, or a combination of such) or stabilized soil. One of the main advantages of asphalt pavements is the economy achieved by using locally available materials.

Generally, it is preferable to treat the granular material used in bases. The most common treatment is to mix asphalt with the granular material, thus producing an asphalt base. Treatment with asphalt is particularly advantageous, as proven at the AASHO Road Test in Illinois (1958-1960), sponsored by the American Association of State Highway Officials (now called American Association of State Highway and Transportation Officials), and administered by the Highway Research Board (now the Transportation Research Board) of the National Academy of Sciences. It was found that a 25 mm (1 in.) thickness of asphalt base was equal in load carrying performance to at least 50 mm (2 in.) or more thickness of granular base materials not treated with asphalt. British road tests also demonstrated this advantage.

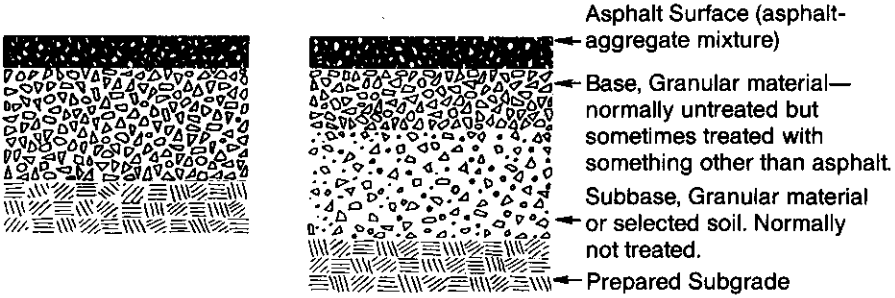
Untreated bases and subbases have been widely used in the past. However, as modern traffic increases in weight and volume, these bases show performance limitations. Consequently, it is becoming common to limit use of untreated bases to pavements designed for lower volumes of lighter traffic.

When the entire pavement structure above the subgrade consists of asphalt mixtures, it is called a Full-Depth® asphalt pavement. This is generally considered the most cost effective, dependable type of pavement for present-day traffic.

Full-Depth Asphalt Pavement


Asphalt Surface
Asphalt Base
Asphalt-aggregate mixture
Asphalt-aggregate mixture, or granular material treated with asphalt.
Prepared Subgrade

Asphalt Pavement With Untreated Base (and Subbase)


Asphalt Surface (asphalt-aggregate mixture)
Base, Granular material—normally untreated but sometimes treated with something other than asphalt.
Subbase, Granular material or selected soil. Normally not treated.
Prepared Subgrade

Asphalt Pavement With Portland Cement Concrete or Combined Portland Cement Concrete and Asphalt Base

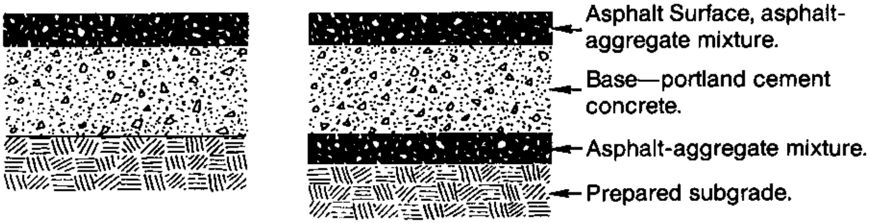

Asphalt Surface, asphalt-aggregate mixture.
Base—portland cement concrete.
Asphalt-aggregate mixture.
Prepared subgrade.

Figure 1.5 Asphalt Pavement Cross-Sections Showing Some Common and Typical Asphalt Pavement Structures.

Other materials sometimes used to treat or stabilize granular base and subbase materials or selected soils are portland cement, lime, lime/fly ash mixture, coal tar, calcium chloride or salt (sodium chloride).

Essential Properties of Asphalt Wearing Surface

The surface of an asphalt pavement exposed to vehicular traffic must be tough to resist distortion and provide a smooth, skid resistant riding surface. It must be waterproof and sloped to shed surface water to the roadside, and thus protect the entire asphalt pavement structure and the subgrade from the weakening effects of moisture. It must resist wear, caused by traffic, and still retain necessary anti-skid properties. It must also be bonded to the layer or course beneath it.

Function of Base and Subbase

Base and subbase are structural elements of the pavement. In conjunction with the overlying asphalt surface, their purpose is to distribute traffic wheel loads over the subgrade or foundation (Figure 1.4). To perform this function, bases and subbases must be built with necessary internal strength properties. In this respect, Full-Depth asphalt pavements have a special advantage over pavements with granular bases. Figure 1.6 for example, shows how wheel load, W, slightly deflects the pavement structure, causing both tensile and compressive stresses within the pavement. Asphalt pavement layers have both tensile and compressive strength to resist these internal stresses. Untreated granular bases have no tensile strength. Therefore, asphalt bases spread the wheel load over broader areas than untreated granular bases. As a result, less total pavement structure thickness is required for an asphalt base. This applies to all loading conditions.

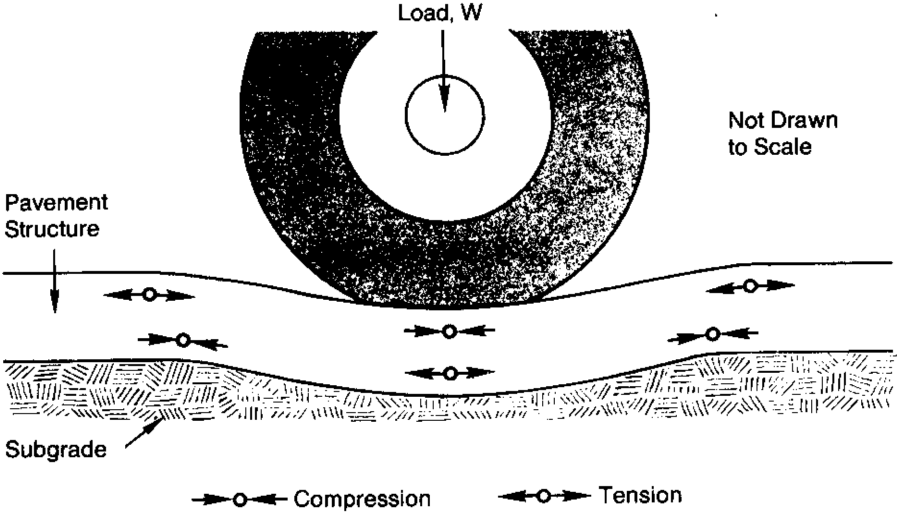


Figure 1.6 Pavement Deflection Results in Tensile and Compressive Stresses in Pavement Structure.

Determining Required Pavement Thickness

A significant advance in highway engineering is the realization and demonstration that structural design of asphalt pavements is similar to the problem of designing any other complex engineering structure. When asphalt pavement was first being introduced, determining the proper thickness was a matter of empiricism and opinion based on experience. Similarly, the same situation once prevailed in determining the dimensions of masonry arches and iron and steel structures. However, these early techniques have long since yielded to engineering analysis. Based on comprehensive analysis of vast volumes of accumulated data, the structural design of asphalt pavement has now been developed into a reliable engineering procedure. Research, aimed at further refinements and a fully rational design procedure, is continuing.

There is no standard thickness for a pavement. Required total thickness is determined by engineering design procedures. Factors considered in the procedures are:

1. Traffic to be served initially and over the design service life of the pavement;
2. Strength and other pertinent properties of the prepared subgrade;
3. Strength and other influencing characteristics of the materials available or chosen for the layers or courses in the total asphalt pavement structure; and
4. Any special factors peculiar to the road being designed, such as environmental factors.

Stage Construction

Because weight and traffic volume normally increase each year, pavement originally built thick enough to handle immediate traffic volumes may not be thick enough and strong enough to handle future needs. With asphalt pavement this problem can be economically met by first building the thickness required, then adding, when needed, layers of asphalt to increase total pavement thickness. This procedure is called *stage construction.* It avoids excessive investment in the beginning; and when a new asphalt pavement layer is added, the wearing surface is equal to or better than the original.

When road-building funds are limited, a strong all-weather asphalt base can be placed and opened to traffic for a year or more. When additional construction funds are available, the asphalt surface course can be added. Similarly, in new housing developments, asphalt base can be placed on all new streets to serve until the development is finished and heavy construction traffic ceases. The asphalt surface is then added after first repairing any damage or settlement suffered by the base. The streets are then at full strength and in an attractive condition appropriate to a new living area.

In building Interstate highways, there have been many examples of planned state construction. Asphalt pavements have been built with sufficient thickness to serve for the first few years, with funds programmed to add strengthening thickness when required by traffic buildup.

Asphalt Usage Guide

Table 1.2 is a suggested guide for the types and grades of asphalt to be used in various pavement construction, maintenance, rehabilitation and recycling procedures.

Types of Asphalt Pavement Construction

Plant Mix

Asphalt paving mixtures prepared in a central mixing facility are traditionally known as plant mixes. Asphalt concrete is considered the highest-quality type of plant mix. It consists of well-graded, high-quality aggregate and asphalt cement. The asphalt and aggregate are heated separately from 121 to 163°C (250 to 325°F), carefully measured and proportioned, then mixed until the aggregate particles are coated with asphalt. The hot mixture, kept hot during transit, is hauled to the construction site, where it is spread on the roadway by an asphalt paving machine. The smooth layer placed by the paver is compacted by rollers to proper density before the asphalt cools.

Asphalt concrete is but one of a variety of hot-mix asphalt (HMA). Other mixes, such as sand asphalt, sheet asphalt, and coarse-graded mixes, are prepared and placed in a similar manner. However, each has one common ingredient—asphalt cement.

Asphalt mixes containing emulsified or cutback asphalt also may be prepared in central mixing facilities. The aggregate may be partially dried and heated or mixed as it is withdrawn from the stockpile.

Asphalt mixtures made with emulsified asphalt and some cutback asphalts can be spread and compacted on the roadway while quite cool. Such mixtures are called cold-laid asphalt plant mixes. They are hauled and placed in normal warm-weather temperatures. To hasten evaporation of emulsification water or cutback solvents, these mixtures, after being placed on the roadway, are sometimes processed or worked back and forth laterally with a motor grader before being spread and compacted.

Mixed-in-Place (Road Mix)

Emulsified asphalt and many cutback asphalts (although the use of cutbacks is declining) are fluid enough to be sprayed onto and mixed into aggregate at moderate- to warm-weather temperatures. When this is done on the area to be paved, it is called *mixed-in-place* construction. Although

Table 1.2 Guide for Uses of Asphalt

Type of Construction	Asphalt Cements																
	Viscosity Graded -Original						Viscosity Graded -Residue						Penetration Graded				
	AC-40	AC-20	AC-10	AC-5	AC-2.5		AR-16000	AR-8000	AR-4000	AR-2000	AR-1000		40-50	60-70	85-100	120-150	200-300
Asphalt-Aggregate Mixtures																	
Asphalt Concrete and Hot Laid Plant Mix																	
Pavement Base and Surfaces																	
Highways	×	×	×	×	×[7]		×	×	×	×	×[7]		×	×	×	×	×[7]
Airports		×	×	×				×	×					×	×	×	
Parking Areas	×	×	×				×	×	×				×	×	×		
Driveways		×	×					×	×					×	×		
Curbs		×						×						×			
Industrial Floors	×	×					×	×					×	×			
Blocks	×						×						×				
Groins	×	×					×	×					×	×			
Dam Facings	×	×					×	×					×	×			
Canal and Reservoir Linings	×	×					×	×					×	×			
Cold-Laid Plant Mix[10]																	
Pavement Base and Surfaces																	
Open-Graded Aggregate																	
Well-Graded Aggregate																	
Patching, Immediate Use																	
Patching, Stockpile																	
Mixed-in-Place (Road Mix)[10]																	
Pavement Base and Surfaces																	
Open-Graded Aggregate																	
Well-Graded Aggregate																	
Sand					×												×
Sandy Soil					×												×
Patching, Immediate Use																	
Patching, Stockpile																	
Recycling																	
Hot-Mix			×	×	×				×	×	×				×	×	×
Cold-Mix[10]					×						×						×
Asphalt-Aggregate Applications																	
Surface Treatments																	
Single Surface Treatment				×	×											×	×
Multiple Surface Treatment				×	×											×	×
Aggregate Seal				×	×					×						×	×
Sand Seal																	
Slurry Seal																	
Asphalt Applications																	
Surface Treatment																	
Fog Seal																	
Prime Coat																	
Tack Coat																	
Dust Laying																	
Mulch																	
Membrane																	
Canal and Reservoir Linings	×													×			
Embankment Envelopes	×	×					×	×						×	×		
Crack Filling																	
Asphalt Pavements																	
Portland Cement Concrete Pavements	×[4]						×[4]							×[4]			

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[1] Mixed-in Prime Only
[2] Diluted with water
[3] Slurry mix
[4] Rubber asphalt compounds
[5] Diluted with water by the manufacturer
[6] MS-2 only
[7] For use in cold climates

Emulsified Asphalts[9]																Cutback-Asphalts									
Anionic									Cationic							Medium Curing (MC)[8]						Slow Curing (SC)			
RS-1	RS-2	MS-1, HFMS-1	MS-2, HFMS-2	MS-2h, HFMS-2h	HFMS-2s	SS-1	SS-1h		CRS-1	CRS-2	CMS-2	CMS-2h	CSS-1	CSS-1h		30	70	250	800	3000		70	250	800	3000
			×	×							×	×													
					×	×	×						×	×				×	×	×			×	×	×
						×	×						×	×				×	×					×	
																		×	×				×	×	
			×	×							×	×							×	×				×	×
					×	×	×						×	×				×	×				×	×	
					×	×	×						×	×			×	×	×						
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			×[1,6]			×[1]	×[1]				×[1]		×[1]	×[1]		×	×	×							
×						×[2]	×[2]		×				×[2]	×[2]											
		×[5]				×[2]	×[2]						×[2]	×[2]		×	×	×				×	×		
						×[2]	×[2]																		
						×[3]	×[3]						×[3]	×[3]											

[8] Before using MC's for spray applications (other than prime coats) check with local pollution control agency

[9] Emulsified asphalts shown are AASHTO and ASTM grades and may not include all grades produced in all geographical areas

[10] Evaluation of emulsified asphalt-aggregate system required to determine the proper grade of emulsified asphalt to use

mixed-in-place is the more general term, and is applicable whether the construction is on a roadway, parking area, or airfield, the term *road mix* is often used when construction is on a roadway.

Mixed-in-place construction can be used for surface, base, or subbase courses. As a surface or wearing course, it usually is satisfactory for light and medium traffic rather than heavy traffic. However, mixed-in-place layers covered by a high-quality asphalt plant-mix surface course make a pavement suitable for heavy traffic service. Advantages of mixing-in-place include:

- Utilization of aggregate already on the roadbed or available from nearby sources and usable without extensive processing.
- Elimination of the need for a central mixing plant. Construction can be accomplished with a variety of machinery often more readily available, such as motor graders, rotary mixer with revolving tines, and traveling mixing plants.

Asphalt Spray Applications

Many necessary and useful purposes are served when paving grade asphalts—temporarily in a fluid condition—can be sprayed in uniform and controlled amounts onto a surface. For example:

Surface Treatments—A sprayed-on application of asphalt to a wearing surface, with or without a thin layer of covering aggregate, is called an *asphalt surface treatment.* By definition such surface treatments are 25 mm (1 in.) or less in thickness. Sometimes these surface treatments are included in original construction, particularly for light duty, farm to market type roads. They are sometimes applied to old pavements after a period of service and before surface deterioration from traffic wear and weathering proceeds too far.

The sprayed-on asphalt serves to improve or restore the waterproof condition of the old pavement surface. Also, it serves to arrest any scuffing or raveling of the wearing surface. The addition of a cover of aggregate over the sprayed-on asphalt restores and improves the skid resistance of the wearing surface.

Multiple surface treatments consist of two or more alternate layers of sprayed-on asphalt and aggregate cover.

Surface treatments that have waterproofing or texture improvement, or both, as their main purposes are called *seal coats.*

Single or multiple surface treatments with aggregate cover also may be placed on granular-surfaced roads to upgrade them for traffic. The treatment eliminates dust, protects the road by shedding water, and provides a smoother riding surface. It is a useful, low-cost, all-weather improvement of a granular-surfaced road, but it has limited traffic capacity and should be used only where traffic is light or where the period of expected service is limited.

Tack Coats and Prime Coats—It is emphasized that each layer in an asphalt pavement should be bonded to the layer beneath. This is accomplished by spraying onto the surface of the underlying layer a thin coating of asphalt (usually emulsified asphalt) to bind the layers together. This thin spread of asphalt is called a tack coat. Tack coats are used to bond asphalt layers to a portland cement concrete base or old asphalt pavements.

When an asphalt pavement or asphalt surface treatment is to be placed on a granular base, it is desirable first to spray on an asphalt (usually cutback asphalt) that will seep into or penetrate the base. This is called *priming,* and the treatment is called *prime coat.* Its purpose is to bind the granular material to the asphalt layer. A prime coat is different from a tack coat as to type and quantity of asphalt used. However, both are spray applications.

There are two other widely used asphalt surface treatments that do not employ spray application procedures. One use is plant mix and the other is a slurry of emulsified asphalt and fine aggregate.

Penetration Macadam—This is an older type of asphalt pavement construction. While many miles of asphalt penetration macadam pavement are still in service, relatively little is used in new construction. Briefly, an asphalt penetration macadam pavement consists of one or more layers of large-sized broken stone and rock chips interlocked by rolling. Fluid asphalt is sprayed onto each layer, and it penetrates the layer to bind the stones together.

An asphalt surface treatment or asphalt mixture of some kind is usually put on the top of a penetration macadam pavement to serve as a wearing surface.

Summary

Proper use of asphalt materials and mixtures is the most cost effective system known for pavement construction. It is important that the engineer-designer have a clear understanding of the different types of asphalt and the purposes of each. Similarly, the engineer should understand the different pavement types—hot mix, cold mix, and surface treatments—and know their specific applications and limitations.

An in-depth understanding of these subjects may require additional studies beyonds the scope of this publication. The Asphalt Institute produces numerous other publications that provide more detailed analysis of individual subjects. Many of these publications are cited at the beginning of each chapter of this manual. In addition, the Institute maintains an engineering staff who can provide assistance on all aspects of design, construction, and maintenance.

Chapter 2
Petroleum Asphalt

This chapter concentrates on asphalt as a material. Properties, characteristics, specifications, and tests are discussed. All of the test details and procedures are not given. Instead, the properties of asphalt, how they are measured, descriptions of the tests and test limits, and specifications for asphalts are discussed.

SECTION 2.1 Refining and Properties of Asphalt Cement
☐ Introduction
☐ Refining Crude Petroleum
☐ Desired Properties/Characteristics of Asphalt Cement
☐ Specifications and Tests for Asphalt Cement

SECTION 2.2 Emulsified and Cutback Asphalt
☐ Introduction
☐ Specifications and Tests for Emulsified Asphalts
☐ Specifications and Tests for Cutback Asphalts

SECTION 2.3 Air-Blown Asphalt
☐ Introduction
☐ Properties and Tests

SECTION 2.4 Control of Asphalt Mixture and Spraying Temperatures
☐ Mixing Temperature
☐ Spraying Temperatures
☐ Precautions

SECTION 2.5 Sampling and Temperature-Volume Relationships, Measurements and Calculations
☐ Sampling
☐ Temperature-Volume Relationships/Measurements and Calculations

SECTION 2.6 Safety Procedures for Hot Asphalt

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VISUAL AIDS

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2. *Testing Asphalt Cement, VA-15,* (132 35mm color slides, narration on cassette, 17 min.), Asphalt Institute.
3. *Testing Emulsified Asphalt, VA-17,* (108 35mm color slides, narration on cassette, 24 min.), Asphalt Institute.
4. *Viscosity Tests of Asphalt, VA-18,* (79 35mm color slides, narration on cassette, 26 min.), Asphalt Institute.

2.1 Refining and Properties of Asphalt Cement

Introduction

According to the American Society for Testing and Materials (ASTM), asphalt is a dark brown to black cementitious material in which the predominating constituents are bitumens that occur in nature or are obtained in petroleum processing. Asphalt is present in varying proportions in most crude petroleums.

Bitumen (according to ASTM) is a class of black or dark-colored (solid, semisolid, or viscous) cementitious substances, natural or manufactured, composed principally of high molecular weight hydrocarbons, of which asphalts, tars, pitches, and asphaltites are typical.

Refining Crude Petroleum

Crude petroleum is composed of a variety of products, including asphalt. Refining separates the various products and allows the asphalt to be recovered. The petroleum asphalt flow chart (Figure 2.1) shows generally, the flow of crude petroleum through a refinery. Emphasized is that part of the process related to the refining and production of apshalt cement.

During the refining process, crude petroleum is fed into a heat exchanger or tube heater, where its temperature is quickly raised for initial distillation. It then enters an atmospheric distillation tower where the lighter (more volatile) components, or fractions, vaporize and are drawn off for further refining into naphtha, gasoline, kerosene, and a wide variety of other petroleum products.

Residue from this atmospheric distillation process is the heavy fraction of the crude petroleum—commonly called topped crude. It may be used as a residual fuel oil, or further processed into a number of products, including asphalt. Vacuum distillation may remove sufficient high boiling fractions to yield a "straight run" asphalt or, if the topped crude contains components of such low volatility that they cannot be removed economically by distillation, solvent extraction of the asphalt or "solvent deasphalting" may be necessary to produce asphalt cement of the desired consistency. Practically, refineries which produce asphalt cements may use all of these processes in combination, and will produce several grades of asphalts which are further blended to produce intermediate grades.

Air-blown asphalts are produced by blowing air at high temperatures through asphalt of the proper consistency. These asphalts are discussed in Section 2.3.

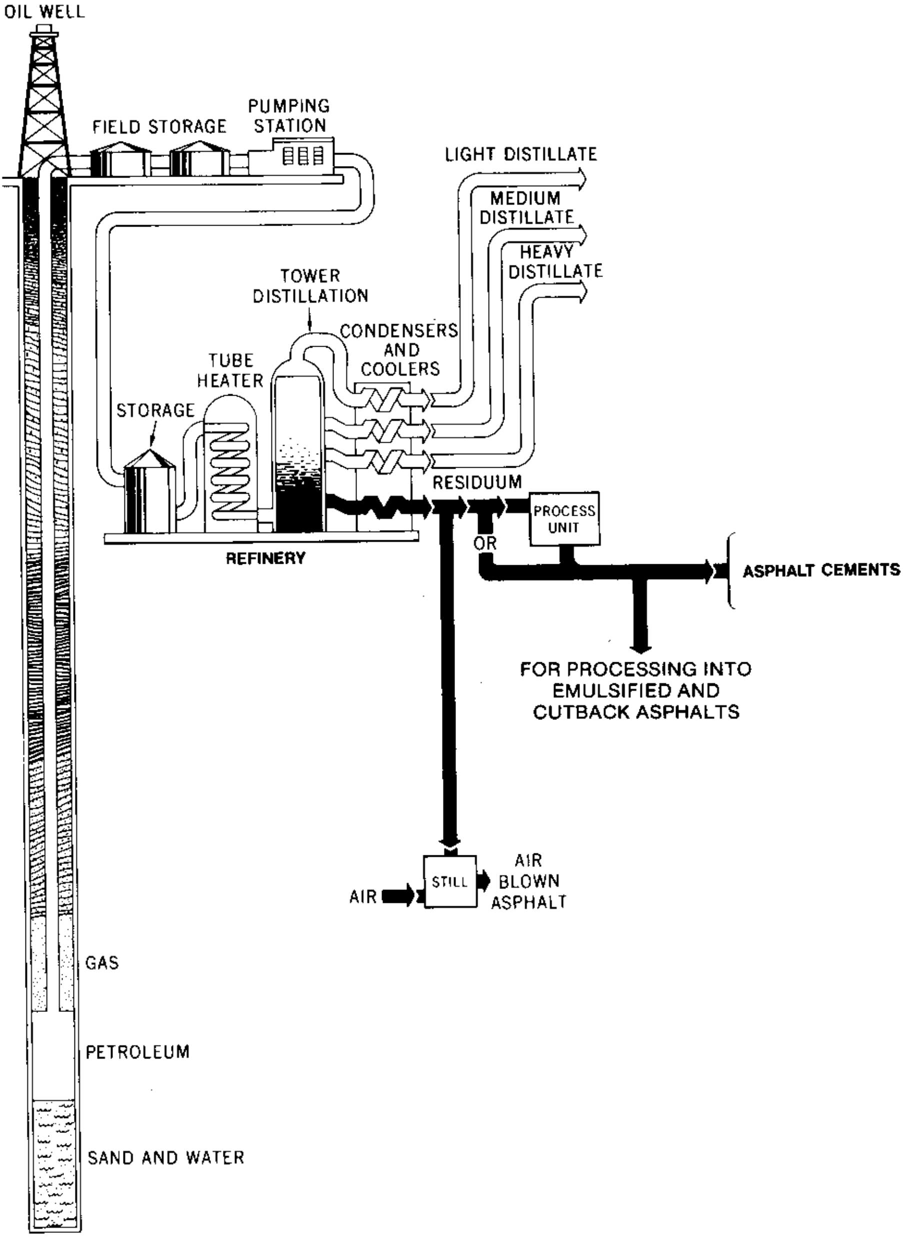

Figure 2.1 Petroleum Asphalt Flow Chart for Asphalt Cement.

Desired Properties or Characteristics of Asphalt Cement

For engineering and construction purposes three properties or characteristics of asphalt are important: (1) consistency (also often called viscosity or fluidity), (2) purity, and (3) safety.

Consistency

Asphalts are thermoplastic materials because they gradually liquefy when heated. They are characterized by their consistency or ability to flow at different temperatures. Consistency is the term used to describe the viscosity or degree of fluidity of asphalt at any particular temperature. The consistency of asphalt cement varies with temperature; therefore, it is necessary to use a standard temperature when comparing the consistency of one asphalt cement with another. Asphalt cements are graded, based on ranges of consistency at a standard temperature.

When asphalt cement is exposed to air in thin films at elevated temperature, e.g., during mixing with aggregate, the asphalt tends to harden. This means that the consistency or viscosity of the asphalt increases. A limited increase is allowable. However, careless temperature and mixing control can cause more damage to an asphalt cement, through hardening, than many years of service on the finished roadway.

Consistency of paving asphalt is commonly specified and measured by a viscosity test or a penetration test. (For air-blown asphalts a softening point test is used.)

Purity

Asphalt cement is composed almost entirely of bitumen, which, by definition, is entirely soluble in carbon disulfide. Refined asphalts are almost pure bitumen and are usually more than 99.5 percent soluble in carbon disulfide. Impurities, if they are present, are inert.

Normally, asphalt cement is free of water or moisture as it leaves the refinery. However, transports loading asphalt may have some moisture present in their tanks. If any water is inadvertently present, it will cause the asphalt to foam when it is heated above 100°C (212°F).

Safety

Asphalt foaming is a safety hazard, and specifications usually require that asphalt not foam at temperatures up to 175°C (347°F).

Asphalt cement, if heated to a high enough temperature, will release fumes that will flash in the presence of a spark or open flame. The temperature at which this occurs is called the flashpoint and is well above the temperatures normally used in paving operations. However, to be sure there is an adequate margin of safety, the flash point of the asphalt is measured and controlled.

Specifications and Tests for Asphalt Cement

Specifications for Asphalt Cements

Asphalt cement is commercially available in several standard ranges of consistency (grades). For many years these ranges were based on measurements by the penetration test only; asphalt cement was available in five standard grades: 40-50, 60-70, 85-100, 120-150, and 200-300, with the numerical grades indicating the allowable ranges of penetration for each grade. The softest (200-300) penetration grade is moderately firm at room temperature; at this temperature gentle finger pressure indents the surface of the sample. The hardest (40-50) penetration grade is of a consistency permitting only a slight thumb print being made under firm pressure when the material is at room temperature.

Grading of asphalts on the basis of the penetration test is an empirical method that is now known to be inadequate with the advent of modern technology. A more fundamental basis is the viscosity test.* Therefore, the modern method is to grade asphalt cements according to their viscosities in poises at 60°C (140°F). The cgs unit of viscosity is 1 g/cm - s (1 dyne - s/cm^2) and is called a poise (P). The SI unit of viscosity is 1 pa - s (1 N - s/m^2) and is equivalent to 10P. There are two series of viscosity grades by which asphalt cement is available. One consists of grades AC-2.5, AC-5, AC-10, AC-20, AC-40, and sometimes AC-30, with the numerical values indicating the viscosity in hundreds of poises at 60°C (140°F). The allowable tolerance for each grade is 20 percent, plus or minus. The other series consists of grades AR-1000, AR-2000, AR-4000, AR-8000, and AR-16000 with the numerical values indicating the viscosity in poises but with the viscosity being measured after the asphalt has been subjected to the rolling thin film oven test. Therefore, the AR series may be interpreted as an "Aged Residue" series. The tolerance on these grades is 25 percent, plus or minus.

Specifications for standard paving grades of asphalt cement have been adopted by the American Association of State Highway and Transportation Officials (AASHTO) and the American Society for Testing and Materials (ASTM). In addition to the viscosity test at 60°C (140°F) or the penetration test for defining the grade or consistency range, several other tests are required to determine specific properties for these products. The tests are listed in Table 2.1, along with the appropriate AASHTO or ASTM test method and indications of which specification requires the test.

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*Because the viscosity test is fundamental and the penetration test is empirical, there is no direct relationship between them. Also, the relationship between penetration and viscosity varies for different asphalts from different crude courses.

Table 2.1 Required Tests for Asphalt Cement in Pavement Construction

			Asphalt Cement Spec.	
Test	Test Method		AASHTO M226[1]	AASHTO M20
	AASHTO	ASTM	Visc. graded	Pen. graded
Viscosity at 60°C (140°F)	T202	D2171	×	----
Viscosity at 135°C (275°F)	T201	D2170	×	----
Penetration	T49	D5	×	×
Flash Point (C.O.C.)	T48	D92	×	×
Thin Film Oven Test	T179	D1754	×	×
Rolling Thin Film Oven Test	T240	D2872	×	----
Ductility	T5	D113	×	×
Solubility in Trichloroethylene	T44	D2042	×	×
Spot Test[2]	T102	----	×	×
Water	T55	D95	×	×

[1] AASHTO M226 and ASTM D3381 have 3 tables of specification limits. The first two are based on original asphalt while Table 3 is based on residue from the rolling thin film oven test.

[2] AASHTO optional test requirement.

Viscosity Tests

Specifications for viscosity-graded asphalt cements normally are based on viscosity ranges at 60°C (140°F). A minimum viscosity at 135°C (275°F) also is usually specified. The purpose is to prescribe limiting values of consistency at these two temperatures. The 60°C (140°F) temperature was chosen as it approximates the maximum temperature of asphalt pavement surfaces in service in the United States and elsewhere in the world where substantial roadbuilding activity is in progress. The 135°C (275°F) temperature was chosen as one that approximates mixing and laydown temperatures for hot asphalt pavements.

The viscosity test at 60°C (140°F) employs a capillary tube viscometer. Two types are in common use. One is the Asphalt Institute vacuum viscometer (Figure 2.2) and the other is the Cannon-Manning vacuum viscometer (Figure 2.3). Each viscometer is calibrated by use of standard calibrating oils. A "calibration factor" is thus developed for each viscometer and used as described later. Normally, viscometers are calibrated by the manufacturer and calibration factors are furnished with them.

The viscometer is mounted in a thermostatically controlled, constant temperature bath (Figure 2.4). Water may be used as the liquid medium in the bath. Preheated asphalt is then poured into the large side of the viscometer until its level reaches the filling line. The filled viscometer is kept in the bath for a prescribed period of time to allow the system to reach an equilibrium temperature of 60°C (140°F).

Because asphalt cement at 60°C (140°F) is too viscous to flow readily through capillary tube viscometers, a partial vacuum is applied to the small side of the viscometer to induce flow. A vacuum control device is illustrated in Figure 2.4. A vacuum pump is also connected into the system.

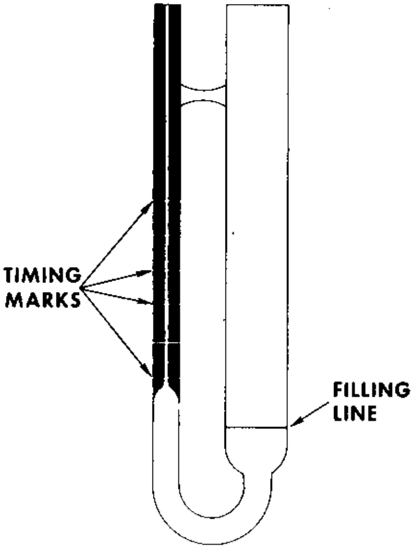

Figure 2.2 The Asphalt Institute Vacuum Viscometer.

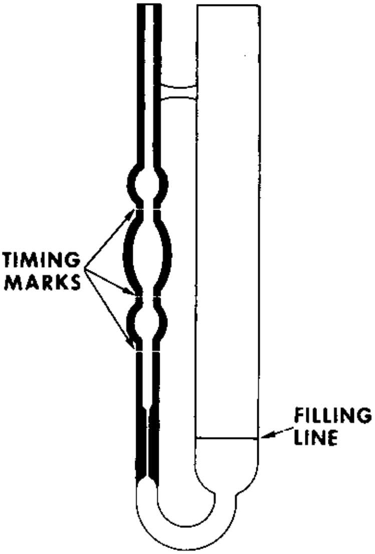

Figure 2.3 Cannon-Manning Vacuum Viscometer.

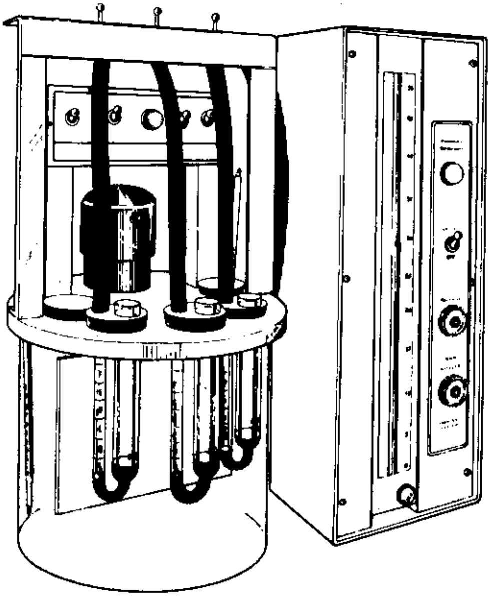

Figure 2.4 Viscometer in Bath.

After the bath, viscometer, and asphalt have stabilized at 60°C (140°F), the prescribed vacuum is applied and the time in seconds required for the asphalt cement to flow between two timing marks is measured by a stop watch. Multiplying this measured time by the calibration factor for the viscometer gives the value for viscosity in poises, the standard unit for measurement of viscosity.

The Asphalt Institute vacuum viscometer has several timing marks. This feature permits it to be used for asphalts of widely varying consistency simply by selecting the appropriate pair of timing marks.

Paving-grade asphalt cements are sufficiently fluid at 135°C (275°F) to flow through capillary tubes under gravitational forces alone. Therefore, a vacuum is not required but a different type of viscometer is used. The one most commonly used is the Zeitfuchs cross-arm viscometer (Figure 2.5). This viscometer is also calibrated with standard calibrating oils.

Since the viscosity tests are conducted at 135°C (275°F), a suitable clear oil must be used as the bath medium. The viscometer is mounted in the bath (Figure 2.6) and asphalt is poured into the large opening until it reaches the

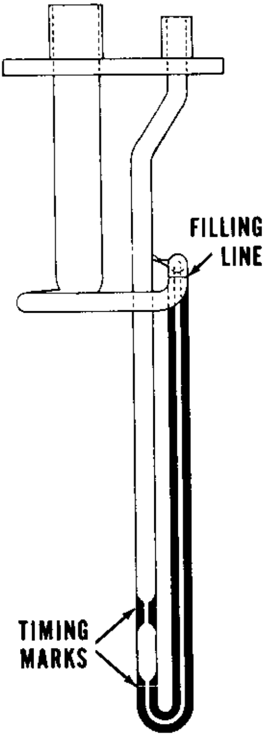

Figure 2.5 Zeitfuchs Cross-Arm Viscometer.

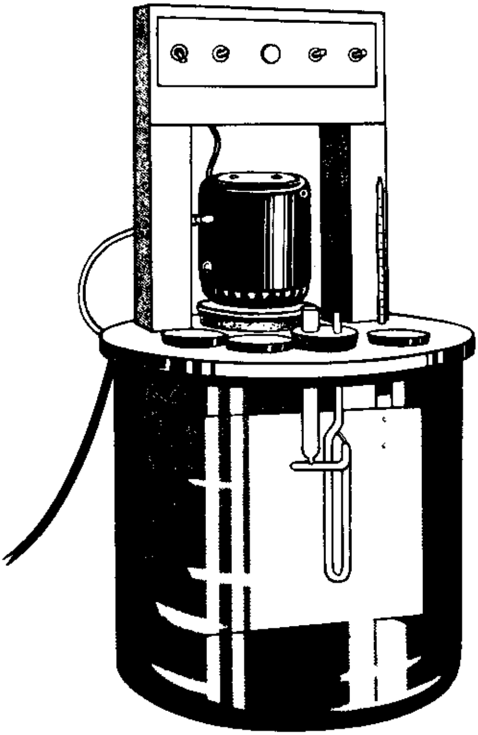

Figure 2.6 Viscometer in Bath.

filling line. The system is then allowed to reach equilibrium temperature as before. A slight pressure is applied to the large opening or a slight vacuum is applied to the small opening to start the asphalt flowing over the siphon section just above the filling line. The asphalt will then flow downward in the vertical section of capillary tubing by gravitational forces. A timer is started when the asphalt reaches the first timing mark and stopped when it reaches the second. The timer interval, multiplied by a calibration factor for the viscometer, gives the kinematic viscosity in centistokes.

Note that viscosity measurements at 135°C (275°F) are expressed in centistokes where those at 60°C (140°F) are in poises. Gravity induces the flow in the kinematic viscosity test (results in centistokes), and density of the material affects the rate of flow through the capillary tube. In the viscosity test where the results are in poises, flow through the capillary tube is induced by a partial vacuum where gravitational effects are negligible. The units of the two tests, poises and stokes or centipoises and centistokes, are related to each other by the density of the asphalt cement.

The viscosity measurements made at the two test temperatures are usually plotted on a viscosity-temperature graph and a line extended between the two points. The slope of this line is an indication of temperature susceptibility. The steeper the slope for a specific asphalt cement, the more it is temperature susceptible. The plotting of this line is important in determining the optimum temperature range for mixing and compacting of hot-mix asphalt. For mix design sample preparation, the Asphalt Institute recommends mixing be done between 150 to 190 centistokes and compaction be accomplished between 250 and 310 centistokes.

Penetration Test

The penetration test is an empirical measure of asphalt consistency. Based on this test, asphalt cements have been classified into standard grades. Figure 2.7 shows the standard penetration test. In testing, a container of asphalt cement is heated to the standard test temperature, 25°C (77°F), in a temperature-controlled water bath. A prescribed needle, weighted to 100 g, is allowed to bear on the surface of the asphalt cement for 5 seconds. The distance, in units of 0.1 mm, that the needle penetrates into the asphalt cement is the penetration measurement.

Occasionally, the penetration test is made at a different temperature. When this occurs, needle load, penetration time, or both, may be varied.

Flash Point Test

When heated, asphalt releases fumes which are combustible. The flash point indicates the temperature at which the asphalt will instantaneously flash in the presence of an open flame. This temperature, however, usually is well below that at which the material will support combustion. The temperature at which the material supports combustion is called the *fire point,* but is seldom used in asphalt specifications.

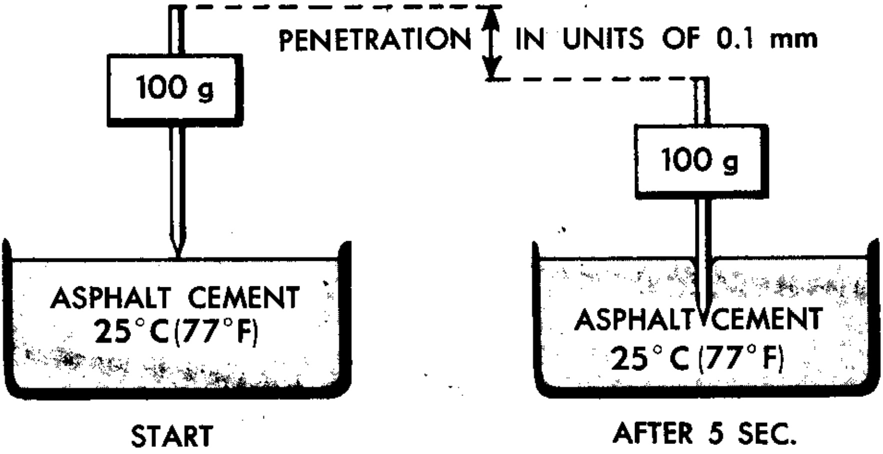

Figure 2.7 Penetration Test.

The flash point of asphalt cement usually is measured by the Cleveland Open Cup (COC) flash point test (Figure 2.8). Here, a brass cup is filled with a specified volume of asphalt and heated at a prescribed rate. A small flame is passed over the surface of the asphalt at prescribed time intervals. The temperature at which sufficient volatiles are released to cause an instantaneous flash is the flash point.

Thin Film Oven Test

The thin film oven (TFO) test actually is not a test, but a procedure intended to subject a sample of asphalt to hardening conditions approximating those that occur in normal, hot-mix facility operations. Mass loss on heating and viscosity or penetration tests, made on the sample before and after the TFO test, are considered to be a measure of the resistance of the material to change under conditions which produce hardening.

The TFO test is made by placing a 50 ml sample of asphalt cement in a weighed, flat-bottomed sample pan 140 mm (5.5 in.) inside diameter and 10 mm (⅜ in.) deep. The asphalt layer is about 3 mm (⅛ in.) deep. The weighed sample and container are placed on a shelf, which rotates approximately 5 to 6 revolutions per minute for 5 hours in a ventilated oven (Figure 2.9) maintained at 163°C (325°F). The mass loss is determined by the loss in weight of the test sample expressed as a percentage of the original weights. The asphalt cement is then poured into a standard container used for the viscosity or penetration test.

Rolling Thin Film Oven Test

A variant of the thin film oven test has been developed by agencies in the western United States in connection with the AR-viscosity grading system. It is known as the rolling thin film oven (RTFO) test. Its purpose is the same as that of the TFO test but equipment and test procedures are different.

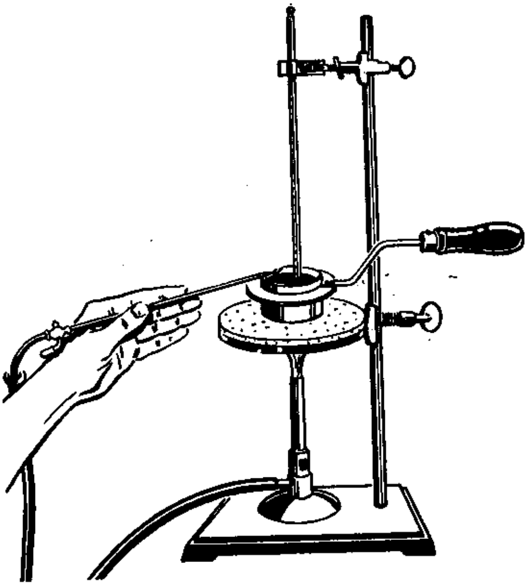

Figure 2.8 Cleveland Open Cup Flash Point Test.

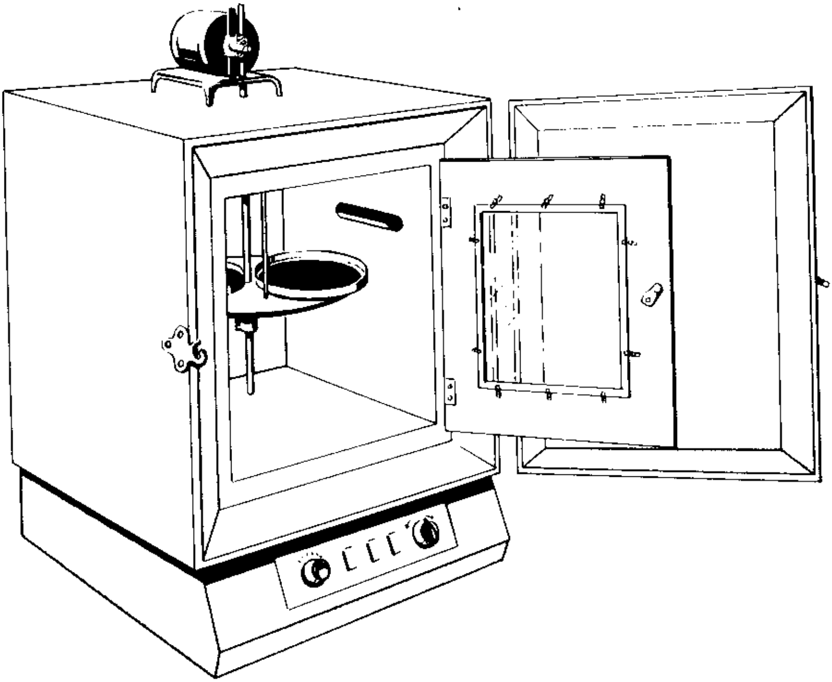

Figure 2.9 Thin Film Oven Test.

The oven used for the RTFO test is illustrated in Figure 2.10 along with a specially-designed bottle used as the container for the test sample. A specified amount of asphalt cement is poured into the bottle, which in turn, is placed in a rack that rotates at a prescribed rate around a horizontal axis, with the oven held at a constant temperature of 163°C (325°F). The rotating bottle continuously exposes fresh films of asphalt cement. Once each rotation, the orifice of the sample bottle passes before an air jet. Heated air from the jet purges accumulated vapors from the sample bottle.

The rolling thin film oven accommodates a larger number of samples than the thin film oven. Time required to achieve a given degree of hardening is less for the RTFO test than for the TFO test. The properties of the RTFO residue determine the grading of the original asphalt in the AR-viscosity graded series.

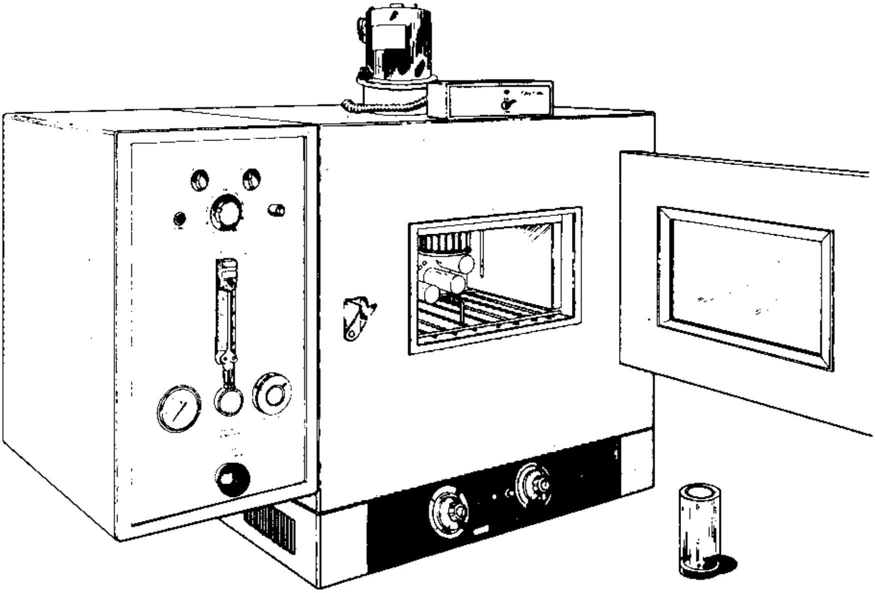

Figure 2.10 Rolling Thin Film Oven Test.

Ductility Test

Ductility is considered an important characteristic of asphalt cements by some engineers. The presence or absence of ductility, however, is usually considered more significant than the actual degree of ductility. Some asphalt cements having an exceedingly high degree of ductility are also more temperature-susceptible. That is, the change in consistency with change in temperatures is large.

Ductility of an asphalt cement is measured by an "extension" type of test (Figure 2.11) where a standard briquette of asphalt cement is molded under standard conditions and dimensions. It is then brought to standard test temperature, normally 25°C (77°F). One part of the briquette is pulled away from the other at a specified rate of speed, normally 5 cm per minute until the thread of asphalt connecting the two parts of sample breaks. The elongation (in centimetres) at which the thread of material breaks is designated as the *ductility* of the asphalt.

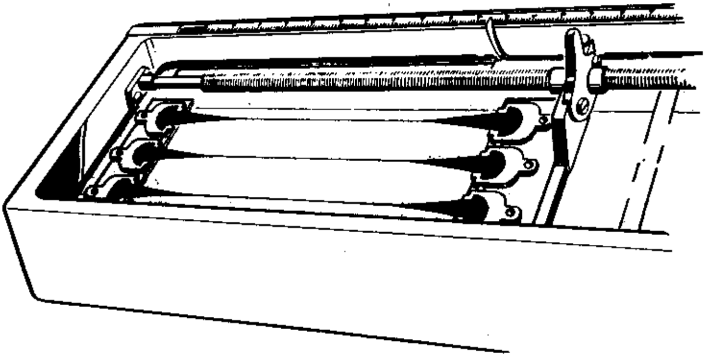

Figure 2.11 Ductility Test.

Solubility Test

The solubility test is a measure of the *purity* of the asphalt cement. The portion of the asphalt cement that is soluble in carbon disulfide represents the active cementing constituents. Only inert matter, such as salts, free carbon, or nonorganic contaminants, are insoluble.

Due to the hazardous nature of carbon disulfide, trichloroethylene, which is also an excellent solvent for asphalt cement, is employed in the solubility test.

Determining solubility is simply a process of dissolving about 2 g of asphalt in a 100 ml of solvent and filtering the solution through a glass fiber pad placed in a porcelain (Gooch) crucible. The amount of material retained on the filter is determined by weighing and is expressed as a percentage of the original sample weight.

2.2 Emulsified and Cutback Asphalts

Introduction

The previous section inferred that heating is one way to liquefy asphalt cement for construction purposes. Asphalt must be liquefied before it can be pumped through pipes, mixed with aggregate, or sprayed through nozzles. When asphalt cement cools, it becomes a semisolid cementing material.

There are other ways, however, to liquefy asphalt for construction operations. It can be dissolved in selected petroleum solvents or emulsified with an emulsifying agent and water. Asphalt liquefied by these methods is known as cutback asphalt and emulsified asphalt.

For cutback asphalts, after completion of construction, the solvent will evaporate*, leaving the asphalt cement to perform its function. With emulsified asphalt, the basic idea is that water will evaporate, leaving the asphalt cement to perform its function.

The petroleum flow chart has been extended in Figure 2.12 to show the flow for making emulsified and cutback asphalt materials. It is important to note that, in each case, asphalt cement is the base material liquefied by emulsifying or cutting back.

It may be noted also that SC-type cutback asphalt may be obtained through direct distillation. This can occur when topped crude is of a suitable quality, and can be refined to proper consistency to serve as one of the standard grades of SC cutback asphalt.

Emulsified Asphalts

In the emulsification process, warm asphalt cement is mechanically milled into minute globules and dispersed in water treated with a small quantity of emulsifying agent. The water is called the continuous phase and the globules of asphalt are called the discontinuous phase. The machine used in this process is a colloid mill, and the asphalt globules produced are extremely small, mostly in the colloidal size range (5-10 microns and smaller).

By proper selection of an emulsifying agent and other manufacturing controls, emulsified asphalts are produced in several types and grades. By choice of emulsifying agent, the emulsified asphalt may be

1. Anionic—asphalt globules are electro-negatively charged;
2. Cationic—asphalt globules are electro-positively charged; and
3. Nonionic—asphalt globules are neutral.

In practice, the first two types are ordinarily used in roadway construction and maintenance. Nonionics, however, are becoming more widely used as emulsion technology advances.

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*Because of solvent evaporation, environmental regulations may restrict or prohibit the use of cutbacks in many areas.

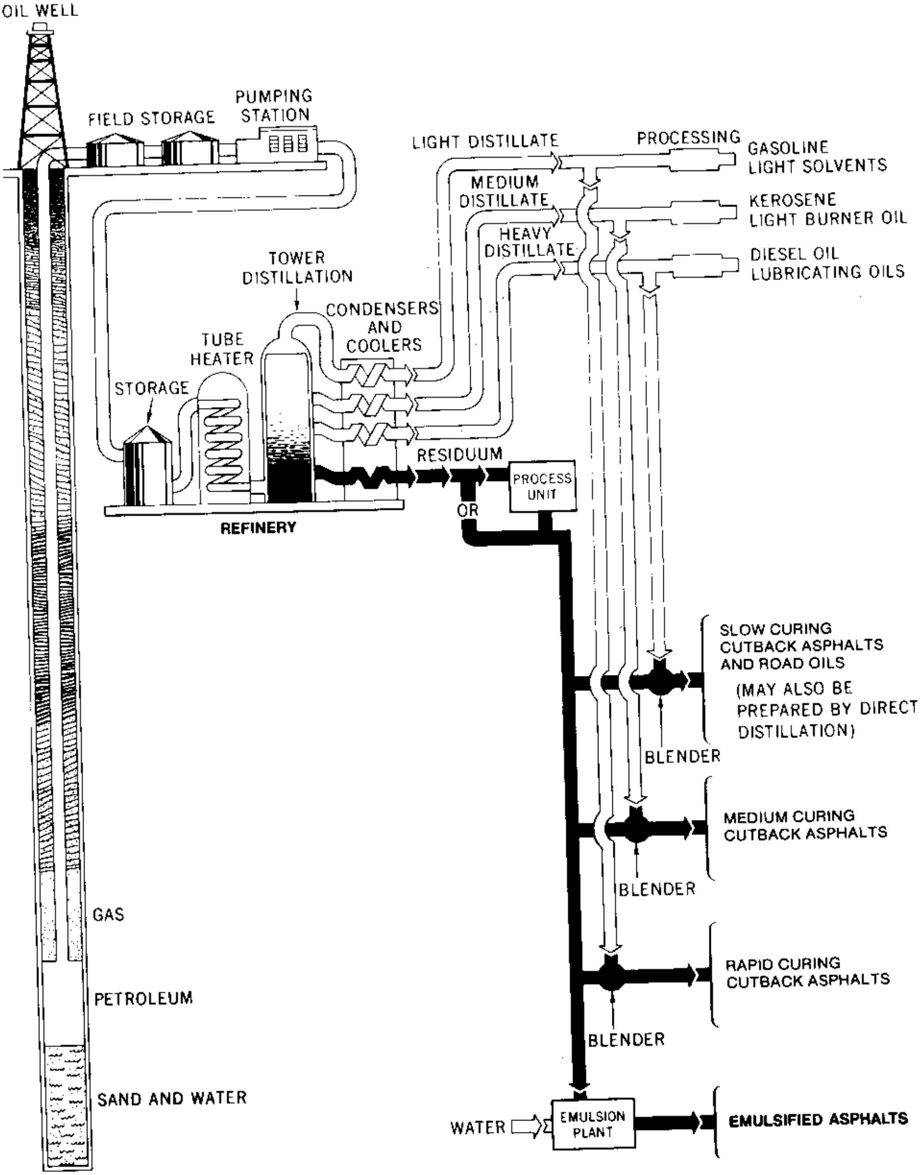

Figure 2.12 Petroleum Asphalt Flow Chart for Emulsified and Cutback Asphalts.

Also by variations in materials and manufacture, emulsified asphalts of both anionic and cationic types are made in several grades:

Anionic	**Cationic**
RS-1	CRS-1
RS-2	CRS-2
MS-1	—
MS-2	CMS-2
MS-2h	CMS-2h
HFMS-1	—
HFMS-2	—
HFMS-2h	—
HFMS-2s	—
SS-1	CSS-1
SS-1h	CSS-1h

The RS, MS or SS given in the type of emulsion is an indication of the emulsion setting rate. The "h" that follows certain grades simply means that a harder base asphalt is used. The "HF" preceding some of the MS grades indicates high-float, as measured by the Float Test (AASHTO T 50 or ASTM D 139). High-float emulsions have a quality, imparted by the addition of certain chemicals, that permits a thicker asphalt film on the aggregate particles with minimum probability of drainage. Some user agencies specify an additional cationic sand-mixing grade designated CMS-2s, which contains more solvent than other cationic grades. All grades in this lengthy list of emulsions may not be stocked by most producers. Communication and planning between user and producer help facilitate service and supply of a given grade.

In order to use emulsions in construction operations, they must remain fluid enough for proper application. The asphalt globules are kept apart by the emulsifying agent until the emulsion has been deposited on the surface of a soil and existing pavement or the aggregate particles. If the asphalt in the emulsion is to perform its ultimate function of cementing and waterproofing, the asphalt phase must separate from the water phase. This occurs through neutralization of the electro-static charges and/or by water evaporation. The asphalt droplets coalesce and produce a continuous film of asphalt on the aggregate or pavement. Coalescence of asphalt droplets occurs more quickly with the rapid setting grades. The time in which the droplets coalesce or separate from the water phase is referred to as the breaking or setting time. The manner and rate at which the emulsions break depends largely on the quantity and properties of the emulsifying agent used in manufacture of the emulsion, and on the relative proportion of the water and asphalt.

Cutback Asphalts

Petroleum solvents used for dissolving asphalt cement are variously called distillate, diluent, or cutter stock. If the solvent used in making the cutback asphalt is highly volatile, it will quickly escape by evaporation. Solvents of lower volatility evaporate more slowly. On the basis of the relative speed of evaporation, cutback asphalts are divided into three types:

1. Rapid-curing (RC)—asphalt cement and a light diluent of high volatility, generally in the gasoline or naphtha boiling point range (RC-70, 250, 800, 3000);
2. Medium-curing (MC)—asphalt cement and medium diluent of intermediate volatility generally in the kerosene boiling point range (MC-30, 70, 250, 800, 3000); and
3. Slow-curing (SC)—asphalt cement and oils of low volatility (SC-70, 250, 800, 3000).

Slow-curing (SC) cutback asphalts are often called *road-oils.* This term originated in earlier days when asphalt residual oil was used to give roads a low-cost, all weather surface.

The degree of liquidity developed in each case depends principally on the proportion of solvent to asphalt cement. To a minor degree, the liquidity of the cutback may be affected by the hardness of the base asphalt from which the cutback is made. The degree of fluidity results in several grades of cutback asphalt—some quite fluid at ordinary temperatures, and others somewhat more viscous. The more viscous grades may require a small amount of heating to make them fluid enough for construction operations.

Uses of Emulsified and Cutback Asphalts

Emulsified asphalts have advantages over hot asphalts in that they can be used with cold as well as heated aggregate, and with aggregate that is dry or damp. The ability to be used with damp aggregate gives emulsified asphalts an advantage over cutback asphalts as well.

Emulsified asphalts are used both for road construction and for many specialty applications. The rapid-setting grades are designed to react quickly with aggregate and revert from the emulsion state to asphalt. They are used primarily for spray applications, such as aggregate (chip) seals, sand seals and similar surface treatments. The RS-2 and CRS-2 grades have high viscosities to prevent runoff. The medium-setting grades are designed for mixing with coarse aggregate. Because these grades do not break immediately upon contact with aggregate, mixes using them remain workable for a few minutes.

They are used extensively in travel plants and mixed-in-place operations. The CMS grades have high viscosities to prevent runoff. The slow-setting grades are designed for maximum mixing stability. They are used with high fines content, dense-graded aggregates. The SS emulsions are generally used for dense-graded aggregate-emulsion bases, soil-asphalt stabilization, asphalt surface mixes, and slurry seals.

Cutback asphalts can be used with cold aggregates, with a minimum of heat. RC and MC types of cutback asphalt were widely used in a variety of roadway, airfield, industrial, and specialty uses. Among the more important uses were road-mixing operations, stockpile mixes, and spray applications (such as prime, tack, and seal coats). But, because the diluents in cutback asphalts are high energy products that are lost by evaporation, and because of environmental regulations, emulsified asphalts have largely supplanted cutbacks for most of these uses. SC-cutback asphalts are used primarily in road-mixing and dust-laying applications. They are also used for stockpile patching mixes, plant-mixing with graded aggregates, and occasionally for priming. Here too, use is declining. It is suggested that local environmental regulations be reviewed prior to using a cutback asphalt.

Specifications and Tests for Emulsified Asphalts

Specifications for Emulsified Asphalts

Emulsified asphalts, which are produced in a variety of viscosities, asphalt cement bases, and setting properties, are manufactured for a number of uses. Specifications for anionic and cationic asphalt emulsions have been adopted by AASHTO and ASTM. For anionic emulsified asphalts, there are three standard types available: rapid-setting (RS), medium-setting (MS), and slow-setting (SS). The same three setting types also are available for cationic emulsified asphalts, designated CRS, CMS, and CSS.

Four grades of high-float medium-setting anionic emulsions, designated HFMS, have been added to standard ASTM specifications. These grades are used primarily in cold and hot plant mixes, coarse aggregate seal coats, and road mixes. High float emulsions have a specific quality that permits a thicker film coating without danger of runoff.

Several tests are required to determine specification compliance of emulsified asphalts.These tests are listed in Table 2.2. Some of the tests for asphalt cement are also required for testing emulsified asphalts. These tests are described in Section 2.1. The other test methods are briefly described.

Saybolt Furol Viscosity Test

The Saybolt Furol viscosity test is used to measure the consistency (rate of flow) properties of emulsified asphalts. As a matter of testing convenience and also to achieve suitable accuracy, the test is performed at one of two temperatures, 25 or 50°C (77 or 122°F), depending on the viscosity characteristics of the specific type and grade of asphalt emulsion.

To conduct the test at 25°C (77°F) a sample is carefully stirred and heated to test temperature. It is then poured through a strainer into a standard tube which is stoppered at the orifice. The stopper is then withdrawn and the time required for 60 ml of asphalt to flow through the orifice is determined (Figure 2.13). The time interval is termed Saybolt Furol viscosity and is measured in seconds. It is apparent that the more viscous the material, the

Table 2.2 **Required Tests for Emulsified Asphalts.**

Test	Type and Specification No.					
	Anionic			Cationic		
Described in AASHTO T59 ASTM D244	AASHTO M140 ASTM D977			AASHTO M208 ASTM D2397		
	RS	MS	SS	CRS	CMS	CSS
Saybolt Furol Viscosity	x	x	x	x	x	x
Storage Stability	x	x	x	x	x	x
Classification[1]	----	----	----	x	----	----
Demulsibility	x	----	----	x	----	----
Coating Ability and Water Resistance	----	x	----	----	x	----
Particle Charge	----	----	----	x	x	x
Cement Mixing	----	----	x	----	----	x
Sieve Test	x	x	x	x	x	x
Distillation	x	x	x	x	x	x
Penetration[2]	x	x	x	x	x	x
Ductility[3]	x	x	x	x	x	x
Sol. in Trichloroethylene[4]	x	x	x	x	x	x

[1] Required only in ASTM D2397. Demulsibility test may be performed in lieu of the classification test.
[2] Test method AASHTO T49 or ASTM D5.
[3] Test method AASHTO T51 or ASTM D113.
[4] Test method AASHTO T44 or ASTM D2402.

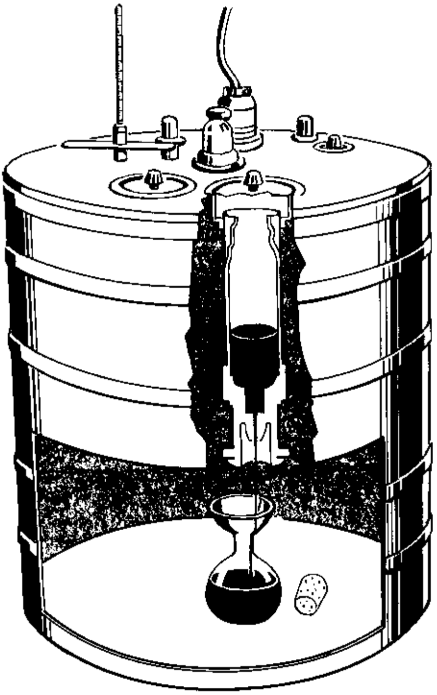

Figure 2.13 Saybolt Furol Viscosity Test.

greater the length of time required for a given volume to flow through the orifice. Thus, an increase in the viscosity number indicates an increase in the viscosity of the asphalt.

To conduct the test at 50°C (122°F), the sample is first heated to between 50 ±3°C (122±5°F) and then poured through the strainer into the tube and brought to test temperature before removing the stopper and timing the flow as already described.

Settlement Test

The settlement test detects the tendency of asphalt globules to "settle out" during storage of emulsified asphalts. It provides the user an element of protection against separation of asphalt and water in unstable asphalt emulsions that may be stored for a period of time.

A 500-ml sample is placed in each of two graduated cylinders, stoppered, and allowed to stand undisturbed for 5 days. Small samples are then taken from the top and bottom parts of each cylinder. Each sample is placed in a beaker and weighed. The samples are then heated until water evaporates; residues are then weighed. The weights obtained provide the basis for determining the difference, if any, between asphalt cement content in the upper and lower portions of the graduated cylinder, thus providing a measure of settlement.

Storage Stability Test

The storage stability test is useful for determining the storage stability of an asphalt emulsion in a comparatively short time. It may be used instead of the 5-day settlement test.

A 500-ml sample is placed in each of two glass cylinders, stoppered, and allowed to stand undisturbed for 24 hours. Fifty-gram samples are taken from the top and bottom parts of each cylinder. Each sample is placed in a beaker, stirred, and weighed. The samples are then heated in an oven to evaporate the water. The residues are then weighed and the average percent residues in the upper and lower portions of the original samples determined. The difference between the upper and lower percentages of residue is recorded as the storage stability.

Classification Test

The classification test is a procedure for distinguishing rapid-setting cationic emulsified asphalt from other types by their failure to coat a sand-cement mixture.

A 461-gram air-dried sample of Ottawa Sand is mixed in a pan with 4 grams of Type III portland cement. A 35 gram sample of emulsified asphalt is added to the blend and is thoroughly mixed with the aggregate for 2½ minutes. At the end of the mixing period any excess emulsion is drained off and a portion of the mix is placed on absorbent paper for visual examination. An excess of uncoated area over coated area of the sand particles is considered as a passing rating for rapid-setting cationic emulsions.

Demulsibility Test

The demulsibility test indicates the relative rate at which the colloidal asphalt globules in the rapid-setting type of emulsified asphalts will break when spread in thin films on soil or aggregate. Calcium chloride causes the minute asphalt globules present in these emulsified asphalts to coalesce. In the test, a solution of calcium chloride and water is thoroughly mixed with the emulsified asphalt; then it is poured over a sieve to determine how much the asphalt globules coalesce.

In testing rapid-setting (RS) emulsions, a very weak solution of calcium chloride and water is employed. Specifications prescribe the concentration of the solution and the minimum amount of asphalt to be retained on the sieve. A high degree of "demulsibility" indicates a rapid-setting (RS) emulsion. It is expected to break almost immediately upon contact with the aggregate on which it is applied.

When cationic emulsions are tested, a solution of sodium dioctyl sulfosuccinate in water is used in place of a calcium chloride solution.

Cement Mixing Test

The cement mixing test is used instead of the demulsibility test for slow-setting grades of emulsified asphalt. It is specified for both the anionic and cationic types to ensure products substantially immune from rapid coalescence of asphalt particles in contact with fine-graded soils or dusty aggregates.

The cement mixing test is performed by stirring in 100 ml of emulsion—diluted with water to 55 percent residue—to 50 grams of high-early-strength portland cement. Additional water is stirred in. The mixture is then washed over a 1.40 mm (No. 14) sieve and the percent of material retained on the sieve is determined.

Coating Ability and Water Resistance

This test has a threefold purpose. It determines the ability of an asphalt emulsion to: (1) coat the aggregate thoroughly, (2) withstand mixing action while remaining as a film on the aggregates and (3) resist the washing action of water after completion of mixing. The test is primarily intended to identify medium-setting asphalt emulsions suitable for mixing with coarse-graded calcareous aggregates. Other aggregates may be used in the test if calcium carbonate is omitted throughout the method. This test is *not* adaptable to rapid-setting or slow-setting asphalt emulsions.

The reference aggregate is coated with calcium carbonate dust and then mixed with the emulsified asphalt. About one-half of the mixture is then placed on absorbent paper for a visual inspection of the surface area of aggregate coated by the emulsified asphalt. The remainder of the mixture is sprayed with water and rinsed until the rinse water runs clear. This material is then placed on absorbent paper and inspected for coating.

A sample of job aggregate is similarly coated with calcium carbonate dust. A given quantity of water is then mixed with the dust-coated aggregate. Emulsified asphalt is added and thoroughly mixed. Inspections are made as described above for the dry-coated aggregates.

Particle Charge Test

This is an identification test for rapid- and medium-setting grades of cationic asphalt emulsions.

A positive electrode (anode) and a negative electrode (cathode) are immersed in a sample of emulsified asphalt (Figure 2.14) and connected to a controlled direct-current electrical source. After 30 minutes, or after the current has dropped to 2 milliamperes, the two electrodes are examined to determine which one has an asphalt deposit. An asphalt deposit on the cathode identifies a cationic emulsified asphalt.

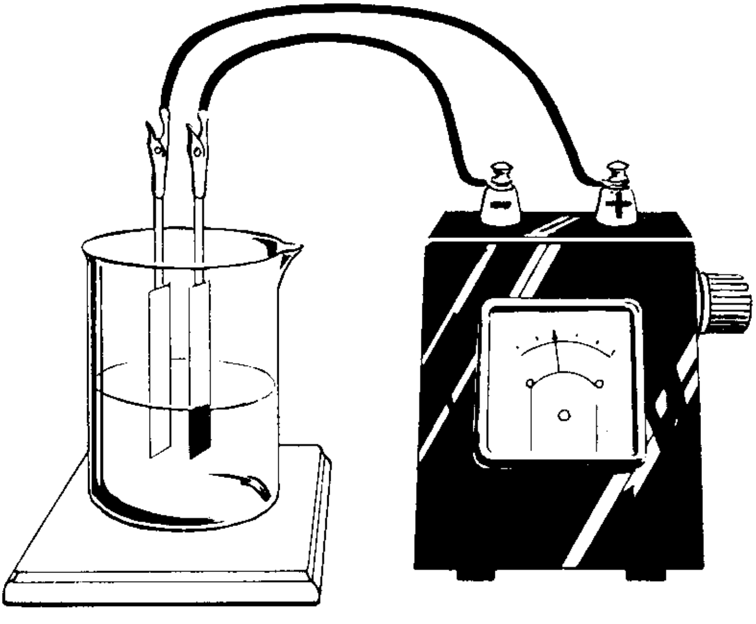

Figure 2.14 Particle Charge Test.

Sieve Test

The sieve test complements the settlement test and has a somewhat similar purpose. It is used to determine quantitatively the percent of asphalt cement present in the form of pieces, strings, or relatively large globules. Such nondispersed particles of asphalt might clog equipment and would tend to provide nonuniform coatings of asphalt on aggregate particles. This nonuniformity might not be detected by the settlement test, which is of value in this regard only when there is a sufficient difference in the specific gravity of asphalt and water to allow settlement.

In the sieve test, 1,000 grams of asphalt emulsion are poured through an 850 μm (No. 20) sieve. For anionic emulsified asphalts, the sieve and retained asphalt are then rinsed with a mild sodium oleate solution. For cationic emulsified asphalts rinsing is with distilled water. After rinsing, the sieve and asphalt are dried in an oven and the relative amount of asphalt retained on the sieve is determined.

Distillation Test

The distillation test is used to determine the relative proportion of asphalt cement and water in the asphalt emulsion. Some grades of emulsified asphalt also contain an oil distillate; the distillation test provides information on the amount of this material in the emulsion. Also, the distillation test provides an asphalt cement residue, on which additional tests (penetration, solubility, and ductility) may be made, as previously described for asphalt cement.

The test procedure is substantially the same as for cutback asphalt. A 200 gram sample of emulsion is distilled to 260°C (500°F). The principal difference in the emulsion distillation test is that the end point of distillation is 260°C (500°F) rather than 360°C (680°F) and an iron or aluminum alloy still and ring burners (Figure 2.15) are used instead of a glass flask and a

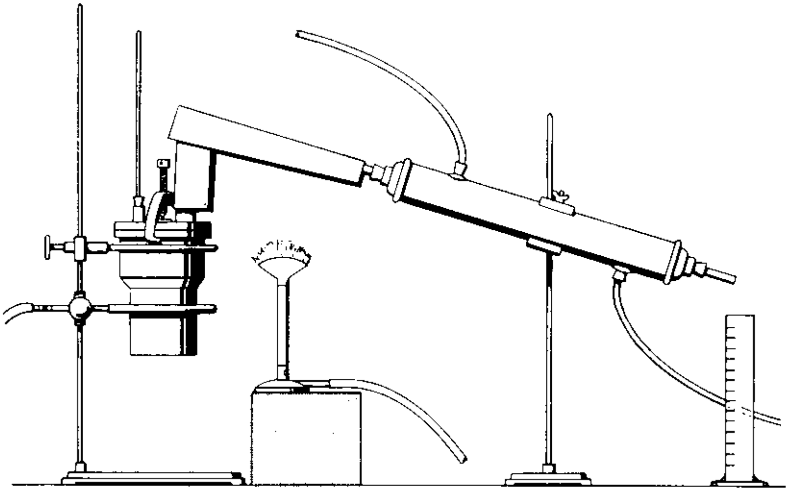

Figure 2.15 Distillation Test for Emulsified Asphalts.

Bunsen burner. This different equipment is designed to prevent trouble that may result from foaming of emulsified asphalt as it is being heated. The end point of distillation is carried to 260°C (500°F), and this temperature is held for 15 minutes in order to produce a smooth, homogeneous residue.

Rapid- and medium-setting grades of cationic asphalt emulsion may include an oily distillate, the maximum amount of which usually is limited by specifications. The distillate collected in the graduated cylinder includes both oil and water from the asphalt emulsion. Because these two materials separate in the graduated cylinder, the amounts of each can be determined.

Specifications and Tests for Cutback Asphalts

Specifications for Cutback Asphalts

Cutback asphalts are available by type and grade. The types (RC, MC, and SC) indicate the relative speed of evaporation and the grades (70, 250, 800, and 3000) indicate the minimum allowable kinematic viscosity in centistokes at 60°C (140°F). One additional grade, MC-30, serves as a special priming grade in some sections of the United States. The maximum allowable viscosity for each grade is twice the minimum allowable value.

The most viscous grades of the three asphalts (RC-3000, MC-3000, and SC-3000) are only moderately less viscous than the lowest viscosity-graded asphalt cement (AC-2.5). The least viscous grades (RC-70, MC-30, MC-70, and SC-70) may be readily poured at room temperature, 25°C(77°F). They have about the same consistency as dairy cream.

Specifications for cutback asphalts and road oils have been adopted by both AASHTO and ASTM. In addition to the viscosity test at 60°C (140°F), several other tests are required to determine specified properties for these products. These tests are listed in Table 2.3 along with the appropriate

Table 2.3 Required Tests for Cutback Asphalts and Road Oils

Test	Test Method		Type and Specification Number		
			AASHTO M81	AASHTO M82	ASTM D2026
	AASHTO	ASTM	RC	MC	SC
Viscosity at 60°C (140°F)	T201	D2170	×	×	×
Flash Point (T.O.C.)	T79	D1310	×	×	----
Flash Point (C.O.C.)	T48	D92	----	----	×
Distillation	T78	D402	×	×	×
Penetration	T49	D5	----	----	----
Ductility	T51	D113	×	×	×
Viscosity at 135°C (275°F)	T202	D2171	×	×	×
Residue of 100 Pen.	T56	D243	----	----	×
Presence of Water	T55	D95	×	×	×
Sol. in Trichloroethylene	T44	D2042	×	×	×
Spot Test[1]	T102	----	×	×	----

[1] AASHTO optional test requirement only.

AASHTO or ASTM test method. Some of the tests for asphalt cement are also required for testing RC, MC, and SC asphalts. The test methods that differ from those on asphalt cement are described below.

Kinematic Viscosity Tests

The kinematic viscosity test is used as a basis for classifying RC, MC, and SC cutback asphalts into standard grades. The test procedure is similar to the kinematic viscosity test for asphalt cement. While the test for asphalt cement is made at 135 °C (275 °F) and requires using oil in a thermostatically controlled bath, it is made at 60 °C (140 °F) for cutback asphalts, with water commonly used as the bath medium. The kinematic viscosity test at 60°C (140°F) is also performed on the SC asphalt residue, following distillation. Except for some differences in sample preparation procedures—mainly to prevent volatiles from escaping—the test is conducted as previously described.

Flash Point Test

The purpose and significance of the flash point test are the same as described earlier for asphalt cement. The flash-point determination for SC materials is made by using the Cleveland open cup, the same test used for asphalt cements. The test procedure for RC and MC grades of cutback asphalts is also substantially the same, except that indirect heating is used because of the volatile nature of diluent in these products. The Tag open cup apparatus (Figure 2.16) is used for this test. The cup is glass, rather than metal, and is heated in a water bath rather than by direct flame.

Cutback asphalts commonly are used at temperatures above their flash point. Rapid-curing cutbacks may flash as low as 27 °C (80 °F). The more volatile the solvent the cutback asphalt contains, the more hazardous its use. All of these materials present some danger in use and must be handled properly.

Distillation Test

As previously indicated, RC, MC, and, in some instances, SC grades of cutback asphalt are blends of asphalt cement and suitable diluents. Properties of these materials are of importance in their application and performance.

The asphalt cement and diluent are separated by the distillation test (Figure 2.17) to determine their quantity and for further identification. Approximately 200 ml of cutback asphalt is poured into a distillation flask connected to a water-cooled condenser tube. As the flask is slowly heated, the diluent vaporizes in the distillation flask and is again liquefied in the condensing tube. The condensate is drained into a graduated cylinder. The quantity of condensate driven off at several specified temperatures is determined, and this is an indication of the volatility characteristics of the diluent. After reaching 360 °C (680 °F), the material remaining in the

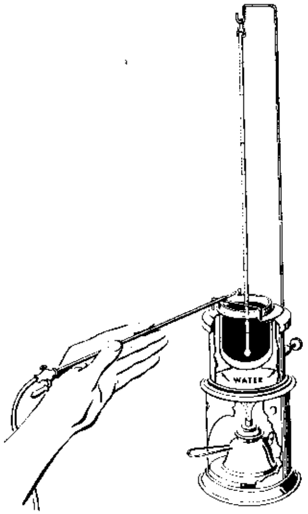

Figure 2.16 Tag Open Cup Test.

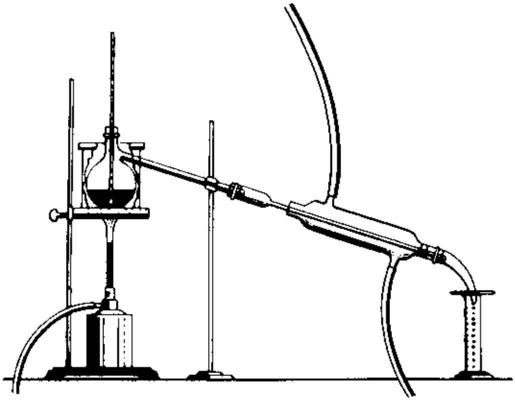

Figure 2.17 Distillation Test for Cutback Asphalts.

distillation flask is considered asphalt cement. For RC and MC cutback asphalts, penetration, ductility, and solubility properties for the residue are then determined as previously described for asphalt cement.

The distillates that evaporate at several temperatures are of little interest for SC cutback asphalts. Distillates evaporating below the specified end point, 360°C (680°F), are largely of an oily nature so that their rate of evaporation in service is quite slow. Therefore, only the total quantity of distillate driven off up to 360°C (680°F) is measured. The residue from distillation to 360°C (680°F) is considered representative of the asphalt portion in the SC cutback asphalt. Its consistency is determined by the kinematic viscosity test.

The solubility test also is run on SC cutback asphalt, but it is performed on the material itself and not on the residue from distillation.

Test for Asphalt Residue of 100 Penetration

This test, made only on SC cutback grades, is perhaps the least significant to the consumer of all tests normally used on these materials. Because the rate of curing in service of an SC cutback asphalt is quite slow, it may or may not reach a penetration of 100 during its service life. The principal value of the test is that it provides a residue upon which the usual tests for asphalt cements may be made.

The test is made by heating sample of SC cutback asphalt from 249 to 260°C (480 to 500°F) and maintaining it at this temperature until it reaches a penetration of 100. The proportionate quantity, by weight, of asphalt residue that remains is determined. Then, a ductility test is made at 25°C (77°F) on the asphalt residue of 100 penetration. The purpose and procedure for this test are as previously described for asphalt cement.

Water Test

Asphalt materials, except for emulsified asphalts, are usually specified to be water free, or substantially so, primarily because water causes foaming when the material is heated, thus creating a hazardous condition.

To determine the amount of water, if any, in cutback asphalt, a measured volume of asphalt is poured into a glass or metal still and thoroughly mixed with a xylene or high-boiling-range petroleum naphtha. A reflux condenser is attached to the still and discharged into a graduated trap. Heat is applied to the still and any water present is collected in the trap. The percentage of water by volume is then calculated.

2.3 Air-Blown Asphalts

Introduction

Special properties may be imported to asphalt by blowing air through the topped crude fraction during the latter part of the refining process. The regular distillation process is discontinued at some point while the topped crude is still liquid. The heavy fraction is then put into a converter and air is blown through it while it is maintained at a high temperature. This process is continued until the asphalt has attained desired properties. Often, such asphalts are called oxidized asphalts. However, this is not strictly a true term because not only oxidation but also vaporization, dehydrogenation, condensation, polymerization and other reactions occur during the air-blowing process.

One of the most significant properties of air-blown asphalt is that the temperature at which asphalt becomes soft is raised. This temperature is usually called the softening point of asphalt. Even at this temperature asphalt still retains its waterproofness and durability. However, there is some reduction in ductility as compared with asphalt cement for paving purposes.

Blown asphalts are used for a wide variety of industrial and specialty purposes. Just a few of these are: a variety of roofing applications, pipe-coating enamels, undersealing asphalts to fill cavities beneath portland cement concrete pavements, and waterproof membranes for lining canals and reservoirs. In many instances blown asphalts are applied in pure form. In other cases, very fine mineral powders (fillers) are mixed with blown asphalt prior to application. Only rarely, if ever, are blown asphalts used in asphalt-aggregate mixtures for paving purposes.

Properties and Tests

Properties and Specifications

Although similar in many respects to the normal paving grades of asphalt cement previously discussed, the blowing process provides materials that soften at higher temperatures than asphalt cements. Because the higher softening point is a most important and desirable property of blown asphalts, they are usually classified in terms of the ring and ball softening point test, rather than viscosity or penetration.

While blown asphalts are graded on the basis of the softening point, there are still penetration test requirements at three temperatures 0 °C, 25 °C, and 46 °C (32 °F, 77 °F, and 115 °F). These requirements provide a degree of control over the temperature susceptibility, or the rate of consistency change with temperature, for these materials.

Specifications have been adopted by AASHTO and ASTM for specialized uses. Although blown asphalts are not normally specified by name, a blowing process is usually required to produce an asphalt that will meet specification requirements. Required tests for some specialized specifications are listed in Table 2.4 along with the appropriate AASHTO or ASTM test method. These tests are briefly described in the following paragraphs, except where they are the same as for asphalt cement and discussed in Section 2.1.

Table 2.4 Required Tests for Some Blown Asphalt Cements

Test	Test Method		Specification and Purpose		
			Undersealing PCC	Dampproofing, Waterproofing	Membrane Linings
	AASHTO	ASTM	AASHTO M238 ASTM D3141	AASHTO M115 ASTM D449	AASHTO M239 ASTM D2521
Softening Point	T53	D36	×	----	----
Flash Point (C.O.C.)	T48	D92	×	×	×
Penetration	T49	D5	×	×	×
Ductility	T51	D113	×	×	×
Loss on Heating	T47	D6	×	×	×
Solubility	T44	D2042	×	×	×

Softening Point Test

The softening point test is used as the basic measurement of consistency for grading blown asphalts. Figure 2.18 shows the test apparatus.

Samples of asphalt loaded with steel balls are confined in brass rings suspended in a beaker of water, glycerine, or ethylene glycol at 25 mm (1 in.) above a metal plate. The liquid is then heated at a prescribed rate. As the asphalt softens, the balls and the asphalt gradually sink toward the plate. At the moment the asphalt touches the plate, the temperature of the water is determined, and this is designated as ring and ball (RB) softening point of asphalt.

Penetration Tests

The specifications indicate penetration tests be performed at:

0°C (32°F) with 200-g needle for 60 s,
25°C (77°F) with 100-g needle for 5 s, and
46 °C (115°F) with 50-g needle for 5s.

The procedure for performing the penetration test as described in Section 2.1 is essentially the same except as noted above.

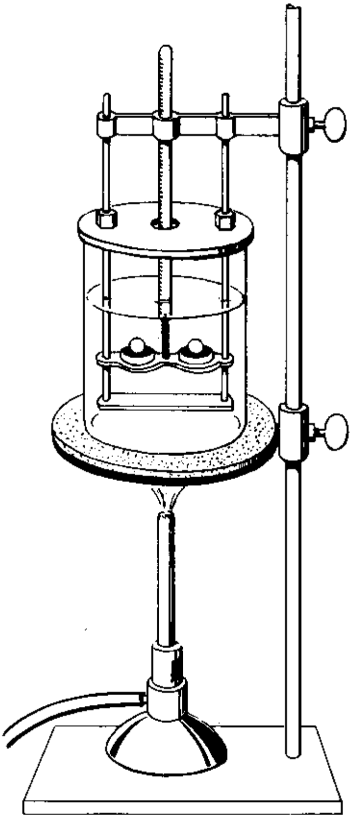

Figure 2.18 Softening Point Test.

Loss on Heating Test

The loss on heating test is generally similar to the thin film oven test as described for asphalt cements. The only differences are in the dimensions of the asphalt sample. Whereas the asphalt sample in the TFO test is about 140 mm (5½ in.) in diameter and 3 mm (⅛ in.) deep, the sample for the loss on heating test is approximately 55 mm (2.2 in.) in diameter and 35 mm (1.4 in.) deep. In both tests the asphalt and container are placed on a rotating shelf in a ventilated oven and maintained at 163°C (325°F) for a period of 5 hours. The shelf rotates at approximately five to six revolutions per minute.

As with the TFO test, the loss on heating also conditions asphalt for other tests. It subjects the asphalt to hardening conditions similar to those expected in the application processes. A penetration test usually is made on the asphalt after the loss on heating test for comparison with the penetration of the asphalt prior to the test.

2.4 Control of Asphalt Mixture and Spraying Temperatures

Asphalt is a thermoplastic material that decreases in viscosity with increasing temperature. The relationship between temperature and viscosity, however, may not be the same for different sources or types and grades of asphalt material.

Mixing Temperature

The mixing temperature for an asphalt mixing facility is governed by placing and compacting requirements. The lowest temperature that will allow time for the mixture to be hauled, placed, and compacted should be used. However, the maximum mixing temperature should never exceed 177°C (350°F).

Both asphalt and aggregate must be heated before they are mixed—the asphalt to make it fluid enough to be pumped and the aggregate to make it dry and hot enough to keep the asphalt fluid while it is coating the particles. The degree of heat in the dried aggregate controls the temperature of the asphalt-aggregate mixture because the temperature of the asphalt cement rapidly adjusts to that of the aggregate when the two are mixed.

Temperatures for emulsified asphalt-aggregate mixtures and MC and SC cutback asphalt-aggregate mixtures are substantially lower than those required for hot-mix asphalt. While these lower mixing temperatures may not provide thoroughly dry aggregates, experience has shown that they can be satisfactory for the intended purpose, especially when asphalt emulsions are used.

Typical temperature ranges for mixing are presented in Tables 2.5 and 2.6.

Spraying Temperatures

The lower temperatures are more critical for spraying asphalt cements and cutback asphalts; therefore, only minimum temperatures are shown in Tables 2.5 and 2.6 for these asphalt materials. The highest temperature used for spraying asphalt cement should be that at which no fogging occurs when the asphalt material leaves the spray nozzle. Tables 2.5 and 2.6 present typical spraying temperature ranges for emulsified asphalts.

Table 2.5 Temperatures for Uses of Asphalt—Degrees Celsius (° C)

Type and Grade of Asphalt	Mixing Temperatures[1]		Spraying Temperatures[5]	
	Dense-Graded Mixes	Open-Graded Mixes	Road Mixes	Surface Treatments
Asphalt Cements				
AC-2.5	115-140	80-120	—	130+
AC-5	120-145	80-120	—	140+
AC-10	120-155	80-120	—	140+
AC-20	130-165	80-120	—	145+
AC-40	130-170	80-120	—	150+
AR-1000	105-135	80-120	—	135+
AR-2000	135-165	80-120	—	140+
AR-4000	135-165	80-120	—	145+
AR-8000	135-165	80-120	—	145+
AR-16000	150-175	80-120	—	—
200-300 pen.	115-150	80-120	—	130+
120-150 pen.	120-155	80-120	—	130+
85-100 pen.	120-165	80-120	—	140+
60-70 pen.	130-170	80-120	—	145+
40-50 pen.	130-175	80-120	—	150+
Emulsified Asphalts				
RS-1	—		—	20-60
RS-2	—		—	50-85
MS-1	10-70[2]		20-70	20-70
MS-2	10-70[2]		20-70	—
MS-2h	10-70[2]		20-70	—
HFMS-1	10-70[2]		20-70	20-70
HFMS-2	10-70[2]		20-70	—
HFMS-2h	10-70[2]		20-70	—
HFMS-2s	10-70[2]		20-70	—
SS-1	10-70[2]		20-70	—
SS-1h	10-70[2]		20-70	—
CRS-1	—		—	50-85
CRS-2	—		—	50-85
CMS-2	10-70[2]		20-70	—
CMS-2h	10-70[2]		20-70	—
CSS-1	10-70[2]		20-70	—
CSS-1h	10-70[2]		20-70	—
Cutback Asphalts (RC, MC, SC)[3]				
30 (MC only)	—	—	—	30+
70	—	—	20+	50+
250	55-80[4]	—	40+	75+
800	75-100[4]	—	55+	95+
3000	80-115[5]	—	—	110+

NOTES: Exact conversions from °F rounded to nearest 5° C.

—Temperatures for asphalt cements and cutback asphalts are guides only.

[1] Temperature of mixture immediately after discharge from the mixing facility rather than temperature of asphalt cement or cutback asphalt.

[2] Temperature of the emulsified asphalt during mixing.

[3] Application temperature may, in some cases, be above the flash point of the material. Caution must therefore be exercised to prevent fire or an explosion.

[4] Rapid-Curing (RC) grades are not recommended for hot mixing.

[5] The maximum temperature (asphalt cement and cutback asphalt) shall be below that at which fogging occurs.

Table 2.6 Temperatures for Uses of Asphalt-Degrees Fahrenheit (° F)

Type and Grade of Asphalt	Mixing Temperatures[1]		Spraying Temperatures[5]	
	Dense-Graded Mixes	Open-Graded Mixes	Road Mixes	Surface Treatments
Asphalt Cements				
AC-2.5	235-280	180-250	—	270+
AC-5	250-295	180-250	—	280+
AC-10	250-315	180-250	—	280+
AC-20	265-330	180-250	—	295+
AC-40	270-340	180-250	—	300+
AR-1000	225-275	180-250	—	275+
AR-2000	275-325	180-250	—	285+
AR-4000	275-325	180-250	—	290+
AR-8000	275-325	180-250	—	295+
AR-16000	300-350	180-250	—	—
200-300 pen.	235-305	180-250	—	265+
120-150 pen.	245-310	180-250	—	270+
85-100 pen.	250-325	180-250	—	280+
60-70 pen.	265-335	180-250	—	295+
40-50 pen.	270-350	180-250	—	300+
Emulsified Asphalts				
RS-1	—		—	70-140
RS-2	—		—	125-175
MS-1	50-160[2]		70-160	70-160
MS-2	50-160[2]		70-160	—
MS-2h	50-160[2]		70-160	—
HFMS-1	50-160[2]		70-160	70-160
HFMS-1h	50-160[2]		70-160	—
HFMS-2h	50-160[2]		70-160	—
HFMS-2s	50-160[2]		70-160	—
SS-1	50-160[2]		70-160	—
SS-1h	50-160[2]		70-160	—
CRS-1	—		—	125-175
CRS-2	—		—	125-175
CMS-2	50-160[2]		70-160	—
CMS-2h	50-160[2]		70-160	—
CSS-1	50-160[2]		70-160	—
CSS-1h	50-160[2]		70-160	—
Cutback Asphalts (RC, MC, SC)[3]				
30 (MC only)	—	—	—	80+
70	—	—	65+	120+
250	135-175[4]	—	105+	165+
800	165-210[4]	—	135+	200+
3000	180-240[4]	—	—	230+

NOTES: See Table 2.5.

Precautions

The purpose of Tables 2.5 and 2.6 is to indicate temperature ranges necessary to provide proper asphalt viscosity for spraying and mixing applications for the grades of asphalt shown. It *must* be recognized, however, that temperature ranges indicated by the charts generally are above the minimum flash point for the RC, MC and SC cutback asphalt materials. In fact, some asphalts will "flash" at temperatures below these indicated ranges. Accordingly, suitable safety precautions are mandatory at all times when handling these cutback asphalts. These safety precautions include, but are not limited to, the following:

(1) Open flames or sparks must not be permitted close to these materials. Controlled heat should be applied in heating kettles, mixers, distributors, or other equipment designed and approved for the purpose.

(2) Open flames must not be used to inspect or examine drums, tank cars, or other containers in which these materials have been stored.

(3) All vehicles transporting these materials must be properly vented.

(4) Only experienced personnel must be permitted to supervise the handling of these materials.

(5) All applicable intrastate and interstate commerce requirements must be met.

2.5 Sampling and Temperature-Volume Relationships, Measurements and Calculations

Sampling

To be meaningful, a sample of shipped asphalt must be truly representative, and must be handled carefully so it does not become contaminated or altered in any way before being tested. If the sample is not truly representative, test results from the sample can be misleading.

Sampling Asphalt from Vehicle Tanks

The best means of obtaining a representative sample of asphalt from a vehicle tank is by using a sampling valve in the truck tank. They are usually located in the lower half of the end bulkhead of the tank. If the sampling valve is used properly, representative samples can be obtained easily and quickly.

The procedure is hardly more than opening the valve, allowing at least 1 litre (a quart) of material to be drained off to remove prior material, and then completely filling the sample container. There are certain precautions to be observed, however, to prevent contamination. For example, the sample containers must be new, clean, and dry, and the filled sample container must be tightly and positively sealed immediately after the sample is obtained. The sample container should not be submerged in solvent, or wiped with a solvent saturated cloth. Any spilled material on the outside of the container should be wiped with a clean, dry cloth immediately after the container has been sealed.

Since the asphalt in most cases is hot, safety precautions are mandatory at all times during sampling. Gloves should be worn, with sleeves rolled down and fastened over the gloves while sampling and sealing containers. Face shields are also a must and no smoking should be permitted. Tongs or some such device should be used to hold the sample container while the sample is being obtained, and the sampler should stand above and away from the sampling valve to the windward side. Care should be taken to prevent the hot material from splashing by taking the sample slowly.

Immediately after filling, sealing, and cleaning as required, the sample containers are marked for identification with a wick marking pencil on the container, not on the lid. Linen tags may be used if they are properly secured to the sample container.

Contamination

It is extremely important that contamination of asphalt products be prevented—from the time they leave the refinery to their incorporation in completed work. This importance also extends to samples and sampling procedures. Tests carried out to ensure compliance with specifications are only meaningful when they are performed on samples that are truly representative of the material used.

Contributing causes for asphalts failing to meet specification requirements can be quite numerous, but contamination is the common culprit in most cases. Investigations have shown that many off-specification samples have flash points too low, penetration too high, viscosities too low, or erratic distillation test results. All of these are typical of solvent-type contaminations. For example, experiments indicate that 0.1 percent diesel oil in asphalt cements may lower the flash point by as much as 28°C (50°F) (Pensky-Martens flash point test), and increase the penetration as much as 10 points. Such contamination amounts to only 19 litres in 19,000 litres (5 gal in 5,000 gal) of asphalt, or 0.001 litre in a litre (0.03 oz in a quart) sample, but the effects on properties of the asphalt are substantial.

Vehicle Tank Conditions for Loading Asphalt

A log showing the material hauled on the previous load must be maintained by the hauler. The log should show whether the tank was cleaned or only drained after the last load. If requested, the hauler should certify that the tank is suitable to load with the product specified. Table 2.7 may be used as a guide or check list. The pump, unloading line, and all piping on the truck must be emptied. Adequate provision must be made for taking a representative sample of the tank contents after loading is completed. A dip sample from the surface of the freshly loaded product is not normally representative.

Temperature-Volume Relationships, Measurements and Calculations

Density (Specific Gravity)

Asphalt has a density (specific gravity) of about 1g/cm^3 (62.4 lb/ft^3) or simply 1.0. There is no specification requirement for density (specific gravity) because, like temperature-viscosity relations, it varies somewhat with different asphalts. However, there often is need to know the exact density (specific gravity) of an asphalt. This value is used, for example, in computing the volume of asphalt at certain temperatures for purposes of purchase payments, or in designing asphalt-aggregate mixtures.

"Density" is the mass per unit volume at 15°C (59°F) expressed in kg/litre. Specific gravity is the ratio of weight of any volume of material to the weight of an equal volume of water, both at a specified temperature, usually 25°C (77°F).

Table 2.7 Guide for Loading Asphalt Products

Last Product in Tank	Product to be Loaded			
	Asphalt Cement	Cutback Asphalt	Cationic Emulsion	Anionic Emulsion
Asphalt Cement	OK to load	OK to load	Empty to no Measurable Quantity	Empty to no Measurable Quantity
Cutback Asphalt	Empty*	OK to load	Empty to no Measurable Quantity	Empty to no Measurable Quantity
Cationic Emulsion	Empty*	Empty to no Measurable Quantity	OK to load	Empty to no Measurable Quantity
Anionic Emulsion	Empty*	Empty to no Measurable Quantity	Empty to no Measurable Quantity	OK to load
Crude Petroleum and residual fuel oils	Empty*	Empty to no Measurable Quantity	Empty to no Measurable Quantity	Empty to no Measurable Quantity
Any product not listed above	Tank must be cleaned	Tank must be cleaned	Tank must be cleaned	Tank must be cleaned

*Any material remaining will produce dangerous conditions.

For asphalt cements and viscous cutback asphalts, the test is usually performed with a pycnometer bottle. By filling the bottle only partially full with asphalt, the asphalt volume is determined by the difference between the volume of the bottle and the volume of the water required to complete the filling. For more fluid asphalts and asphalt emulsions, a hydrometer is used.

To indicate definite conditions applicable to a given specific gravity value, the temperature of the asphalt and water should be shown. Thus a specific gravity at 15.6/15.6°C (60/60°F) indicates that the determination was made with both materials at a temperature of 15.6°C (60°F). The specific gravity of asphalt at 15.6/15.6°C (60/60°F) can be calculated knowing the specific gravity at 25/25°C (77/77°F). This procedure is explained in the following paragraphs.

Temperature-Volume Relationships and Calculations

All liquids and most solids undergo changes in volume with changes in temperature. They expand when heated and contract when cooled. The change in unit volume per degree change in temperature is termed *coefficient of expansion,* a factor that varies with variations in the density (specific gravity) of the asphalt product.

Temperature-volume correction tables for asphalt materials are published in ASTM D4311 and are reproduced here in Tables 2.8 (°C) and 2.9 (°F). Two columns of correction factors are given in each table. The selection of the appropriate column (A or B) is dependent upon the specific gravity or density of the asphalt, as indicated in the table footnotes. The column A factors apply to the majority of asphalts. Temperature-volume relationships for emulsified asphalts are given in Table 2.10.

The multipliers given in the tables are used to convert a known volume at a given temperature to volume at 15° C (60° F), which is customarily used as the standard basis for volume determinations of asphalt.

$$V = V_T M_T \qquad (1)$$

where V = volume at 15° C or 60° F,
V_T = volume at given temperature, and
M_T = multiplier from appropriate table

Example: (U.S. Customary Units)

The specific gravity of an asphalt product is found to be 0.985 at 60°F. The volume of this material is measured to be 9,000 gal at 180°F. The column A factors of Table 2.9 apply since the specific gravity exceeds 0.967. For a temperature of 180°F the correction factor, M_T, is 0.9587. Thus, the volume of the material at 60°F is 9,000 x (0.9587) = 8,628 gal.

Specific Gravity Calculations

The basic formula for specific gravity of a substance is the mass in air of a unit volume of the substance at a stated temperature divided by the mass in air of an equal volume of gas-free distilled water at a stated temperature. Or:

$$G_x = \frac{W_x}{W_w} \quad (2a) \qquad or \qquad G_x = \frac{W_x}{V_x \gamma_w} \quad (2b)$$

where G_x = specific gravity of the substance
W_x = mass of a unit volume of the substance
W_w = mass of a unit volume of gas-free distilled water
V_x = volume of the substance
γ_w = density of water

The volume of asphalt, which changes with temperature, can be determined using Eq. 1. However, the volume of water (or density) also changes with temperature, as discussed in the following paragraphs.

Table 2.8 Temperature-Volume Corrections for Asphalt Materials (Degrees Celsius)

Observed Temperature, °C	Volume Correction[C,D] Factor to 15°C		Observed Temperature, °C	Volume Correction[C,D] Factor to 15°C		Observed Temperature, °C	Volume Correction[C,D] Factor to 15°C		Observed Temperature, °C	Volume Correction[C,D] Factor to 15°C	
	A	B		A	B		A	B		A	B
−25,0	1,0254	1,0290	12,5	1,0016	1,0018	50,0	0,9782	0,9752	87,5	0,9552	0,9492
−24,5	1,0251	1,0286	13,0	1,0012	1,0014	50,5	0,9779	0,9749	88,0	0,9548	0,9489
−24,0	1,0248	1,0283	13,5	1,0009	1,0014	51,0	0,9776	0,9745	88,5	0,9545	0,9485
−23,5	1,0244	1,0279	14,0	1,0006	1,0007	51,5	0,9773	0,9742	89,0	0,9542	0,9482
−23,0	1,0241	1,0276	14,5	1,0003	1,0004	52,0	0,9770	0,9738	89,5	0,9539	0,9478
−22,5	1,0238	1,0272	15,0	1,0000	1,0000	52,5	0,9767	0,9735	90,0	0,9536	0,9475
−22,6	1,0235	1,0268	15,5	0,9997	0,9998	53,0	0,9763	0,9731	90,5	0,9533	0,9472
−21,5	1,0232	1,0265	16,0	0,9994	0,9993	53,5	0,9760	0,9728	91,0	0,9530	0,9468
−21,0	1,0228	1,0261	16,5	0,9991	0,9989	54,0	0,9757	0,9724	91,5	0,9527	0,9465
−20,5	1,0225	1,0258	17,0	0,9988	0,9986	54,5	0,9754	0,9721	92,0	0,9524	0,9461
−20,0	1,0222	1,0254	17,5	0,9985	0,9982	55,0	0,9751	0,9717	92,5	0,9521	0,9458
−19,5	1,0219	1,0250	18,0	0,9981	0,9978	55,5	0,9748	0,9714	93,0	0,9518	0,9455
−19,0	1,0216	1,0247	18,5	0,9978	0,9975	56,0	0,9745	0,9710	93,5	0,9515	0,9451
−18,5	1,0212	1,0243	19,0	0,9975	0,9971	56,5	0,9742	0,9707	94,0	0,9512	0,9448
−18,0	1,0209	1,0239	19,5	0,9972	0,9968	57,0	0,9739	0,9703	94,5	0,9509	0,9444
−17,5	1,0206	1,0236	20,0	0,9969	0,9964	57,5	0,9736	0,9700	95,0	0,9506	0,9441
−17,0	1,0203	1,0232	20,5	0,9966	0,9961	58,0	0,9732	0,9696	95,5	0,9503	0,9438
−16,5	1,0200	1,0228	21,0	0,9963	0,9957	58,5	0,9729	0,9693	96,0	0,9500	0,9434
−16,0	1,0196	1,0224	21,5	0,9959	0,9954	59,0	0,9726	0,9689	96,5	0,9497	0,9431
−15,5	1,0193	1,0221	22,0	0,9956	0,9950	59,5	0,9723	0,9686	97,0	0,9494	0,9427
−15,0	1,0190	1,0217	22,5	0,9953	0,9947	60,0	0,9720	0,9682	97,5	0,9491	0,9424
−14,5	1,0187	1,0213	23,0	0,9950	0,9943	60,5	0,9717	0,9679	98,0	0,9488	0,9421
−14,0	1,0184	1,0210	23,5	0,9947	0,9940	61,0	0,9714	0,9675	98,5	0,9485	0,9417
−13,5	1,0180	1,0206	24,0	0,9943	0,9936	61,5	0,9711	0,9672	99,0	0,9482	0,9414
−13,0	1,0177	1,0203	24,5	0,9940	0,9933	62,0	0,9708	0,9668	99,5	0,9479	0,9410
−12,5	1,0174	1,0199	25,0	0,9937	0,9929	62,5	0,9705	0,9665	100,0	0,9476	0,9407
−12,0	1,0171	1,0195	25,5	0,9937	0,9925	63,0	0,9701	0,9661	100,5	0,9473	0,9404
−11,5	1,0168	1,0192	26,0	0,9934	0,9922	63,5	0,9698	0,9658	101,0	0,9470	0,9400
−11,0	1,0164	1,0188	26,5	0,9928	0,9918	64,0	0,9695	0,9654	101,5	0,9467	0,9397
−10,5	1,0161	1,0185	27,0	0,9925	0,9915	64,5	0,9692	0,9651	102,0	0,9464	0,9393
−10,0	1,0158	1,0181	27,5	0,9922	0,9911	65,0	0,9689	0,9647	102,5	0,9461	0,9390
−9,5	1,0155	1,0177	28,0	0,9918	0,9907	65,5	0,9686	0,9644	103,0	0,9458	0,9387
−9,0	1,0152	1,0174	28,5	0,9915	0,9904	66,0	0,9683	0,9640	103,5	0,9455	0,9383
−8,5	1,0148	1,0170	29,0	0,9912	0,9900	66,5	0,9680	0,9637	104,0	0,9452	0,9380
−8,0	1,0145	1,0166	29,5	0,9909	0,9897	67,0	0,9677	0,9633	104,5	0,9449	0,9376
−7,5	1,0142	1,0163	30,0	0,9906	0,9893	67,5	0,9674	0,9630	105,0	0,9446	0,9373
−7,0	1,0139	1,0159	30,5	0,9903	0,9889	68,0	0,9670	0,9626	105,5	0,9443	0,9370
−6,5	1,0136	1,0155	31,0	0,9900	0,9886	68,5	0,9667	0,9623	106,0	0,9440	0,9366
−6,0	1,0132	1,0151	31,5	0,9897	0,9882	69,0	0,9664	0,9619	106,5	0,9437	0,9363
−5,5	1,0129	1,0148	32,0	0,9894	0,9879	69,5	0,9661	0,9616	107,0	0,9434	0,9359
−5,0	1,0126	1,0144	32,5	0,9891	0,9875	70,0	0,9568	0,9612	107,5	0,9431	0,9356
−4,5	1,0123	1,0140	33,0	0,9887	0,9871	70,5	0,9655	0,9609	108,0	0,9428	0,9353
−4,0	1,0120	1,0137	33,5	0,9984	0,9868	71,0	0,9652	0,9605	108,5	0,9425	0,9349
−3,5	1,0117	1,0133	34,0	0,9881	0,9864	71,5	0,9649	0,9602	109,0	0,9422	0,9346
−3,0	1,0114	1,0130	34,5	0,9878	0,9861	72,0	0,9646	0,9598	109,5	0,9419	0,9342
−2,5	1,0111	1,0126	35,0	0,9875	0,9857	72,5	0,9643	0,9595	110,0	0,9416	0,9339
−2,0	1,0107	1,0122	35,5	0,9872	0,9854	73,0	0,9640	0,9592	110,5	0,9413	0,9336
−1,5	1,0104	1,0119	36,0	0,9869	0,9850	73,5	0,9637	0,9588	111,0	0,9410	0,9932
−1,0	1,0101	1,0115	36,5	0,9866	0,9847	74,0	0,9634	0,9585	111,5	0,9407	0,9329
−0,5	1,0098	1,0112	37,0	0,9863	0,9843	74,5	0,9631	0,9581	112,0	0,9404	0,9325
0	1,0095	1,0108	37,5	0,9860	0,9840	75,0	0,9628	0,9578	112,5	0,9401	0,9322
0,5	1,0092	1,0104	38,0	0,9856	0,9836	75,5	0,9625	0,9575	113,0	0,9397	0,9319
1,0	1,0089	1,0101	38,5	0,9853	0,9833	76,0	0,9622	0,9571	113,5	0,9394	0,9315
1,5	1,0085	1,0097	39,0	0,9850	0,9829	76,5	0,9619	0,9568	114,0	0,9391	0,9312
2,0	1,0082	1,0094	39,5	0,9847	0,9826	77,0	0,9616	0,9564	114,5	0,9388	0,9308
2,5	1,0079	1,0090	40,0	0,9844	0,9822	77,5	0,9613	0,9561	115,0	0,9385	0,9305
3,0	1,0076	1,0086	40,5	0,9841	0,9819	78,0	0,9609	0,9557	115,5	0,9382	0,9302
3,5	1,0073	1,0083	41,0	0,9838	0,9815	78,5	0,9606	0,9554	116,0	0,9379	0,9298
4,0	1,0069	1,0079	41,5	0,9835	0,9812	79,0	0,9603	0,9550	116,5	0,9376	0,9295
4,5	1,0066	1,0076	42,0	0,9832	0,9808	79,5	0,9600	0,9547	117,0	0,9373	0,9292
5,0	1,0063	1,0072	42,5	0,9829	0,9805	80,0	0,9597	0,9543	117,5	0,9371	0,9289
5,5	1,0060	1,0068	43,0	0,9825	0,9801	80,5	0,9594	0,9540	118,0	0,9368	0,9285
6,0	1,0057	1,0065	43,5	0,9822	0,9798	81,0	0,9591	0,9536	118,5	0,9365	0,9282
6,5	1,0053	1,0061	44,0	0,9819	0,9794	81,5	0,9588	0,9533	119,0	0,9362	0,9279
7,0	1,0050	1,0058	44,5	0,9816	0,9791	82,0	0,9585	0,9529	119,5	0,9359	0,9275
7,5	1,0047	1,0054	45,0	0,9813	0,9787	82,5	0,9582	0,9526	120,0	0,9356	0,9272
8,0	1,0044	1,0050	45,5	0,9810	0,9784	83,0	0,9578	0,9523	120,5	0,9353	0,9269
8,5	1,0041	1,0047	46,0	0,9807	0,9780	83,5	0,9576	0,9519	121,0	0,9350	0,9265
9,0	1,0037	1,0043	46,5	0,9804	0,9777	84,0	0,9573	0,9516	121,5	0,9347	0,9262
9,5	1,0034	1,0040	47,0	0,9801	0,9773	84,5	0,9570	0,9512	122,0	0,9344	0,9258
10,0	1,0031	1,0036	47,5	0,9798	0,9770	85,0	0,9567	0,9509	122,5	0,9341	0,9255
10,5	1,0028	1,0032	48,0	0,9794	0,9766	85,5	0,9564	0,9506	123,0	0,9338	0,9252
11,0	1,0025	1,0029	48,5	0,9791	0,9763	86,0	0,9561	0,9502	123,5	0,9335	0,9248
11,5	1,0022	1,0025	49,0	0,9788	0,9759	86,5	0,9558	0,9499	124,0	0,9332	0,9245
12,0	1,0019	1,0022	49,5	0,9785	0,9756	87,0	0,9555	0,9495	124,5	0,9329	0,9241

[C] Use column A factors for asphalts with density at 15°C of 966 kg/m^3 or higher.

[D] Use column B factors for asphalts with density at 15°C of 850 to 965 kg/m^3.

Table 2.8 (Continued) Temperature-Volume Corrections for Asphalt Materials (Degrees Celsius)

Observed Temperature, °C	Volume Correction^{C,D} Factor to 15°C		Observed Temperature, °C	Volume Correction^{C,D} Factor to 15°C		Observed Temperature, °C	Volume Correction^{C,D} Factor to 15°C		Observed Temperature, °C	Volume Correction^{C,D} Factor to 15°C	
	A	B		A	B		A	B		A	B
125,0	0,9326	0,9238	162,5	0,9104	0,8991	200,0	0,8886	0,8749	237,5	0,8673	0,8514
125,5	0,9323	0,9235	163,0	0,9101	0,8987	200,5	0,8883	0,8746	238,0	0,8670	0,8510
126,0	0,9320	0,9231	163,5	0,9098	0,8984	201,0	0,8880	0,8743	238,5	0,8667	0,8507
126,5	0,9317	0,9228	164,0	0,9095	0,8981	201,5	0,8877	0,8739	239,0	0,8664	0,8504
127,0	0,9314	0,9225	164,5	0,9092	0,8977	202,0	0,8874	0,8736	239,5	0,8661	0,8501
127,5	0,9311	0,9222	165,0	0,9089	0,8974	202,5	0,8872	0,8733	240,0	0,8658	0,8498
128,0	0,9308	0,9218	165,5	0,9086	0,8971	203,0	0,8869	0,8730	240,5	0,8655	0,8495
128,5	0,9305	0,9215	166,0	0,9083	0,8968	203,5	0,8866	0,8727	241,0	0,8652	0,8492
129,0	0,9302	0,9212	166,5	0,9080	0,8964	204,0	0,8863	0,8723	241,5	0,8650	0,8489
129,5	0,9299	0,9208	167,0	0,9077	0,8961	204,5	0,8860	0,8720	242,0	0,8647	0,8486
130,0	0,9296	0,9205	167,5	0,9075	0,8958	205,0	0,8857	0,8717	242,5	0,8644	0,8483
130,5	0,9293	0,9202	168,0	0,9072	0,8955	205,5	0,8854	0,8714	243,0	0,8641	0,8480
131,0	0,9290	0,9198	168,5	0,9069	0,8952	206,0	0,8851	0,8711	243,5	0,8638	0,8477
131,5	0,9287	0,9195	169,0	0,9066	0,8948	206,5	0,8849	0,8708	244,0	0,8636	0,8474
132,0	0,9284	0,9191	169,5	0,9063	0,8945	207,0	0,8846	0,8705	244,5	0,8633	0,8471
132,5	0,9281	0,9188	170,0	0,9060	0,8942	207,5	0,8843	0,8702	245,0	0,8630	0,8468
133,0	0,9278	0,9185	170,5	0,9057	0,8939	208,0	0,8840	0,8698	245,5	0,8627	0,8465
133,5	0,9275	0,9181	171,0	0,9054	0,8935	208,5	0,8837	0,8695	246,0	0,8624	0,8462
134,0	0,9272	0,9178	171,5	0,9051	0,8932	209,0	0,8835	0,8692	246,5	0,8622	0,8459
134,5	0,9269	0,9174	172,0	0,9048	0,8929	209,5	0,8832	0,8689	247,0	0,8619	0,8456
135,0	0,9266	0,9171	172,5	0,9046	0,8926	210,0	0,8829	0,8686	247,5	0,8616	0,8453
135,5	0,9263	0,9168	173,0	0,9043	0,8922	210,5	0,8826	0,8683	248,0	0,8613	0,8449
136,0	0,9260	0,9164	173,5	0,9040	0,8919	211,0	0,8823	0,8680	248,5	0,8610	0,8446
136,5	0,9257	0,9161	174,0	0,9037	0,8916	211,5	0,8820	0,8676	249,0	0,8608	0,8443
137,0	0,9254	0,9158	174,5	0,9034	0,8912	212,0	0,8817	0,8673	249,5	0,8605	0,8440
137,5	0,9251	0,9155	175,0	0,9031	0,8909	212,5	0,8815	0,8670	250,0	0,8602	0,8437
138,0	0,9248	0,9151	175,5	0,9028	0,8906	213,0	0,8812	0,8667	250,5	0,8599	0,8434
138,5	0,9246	0,9148	176,0	0,9025	0,8903	213,5	0,8809	0,8664	251,0	0,8596	0,8431
139,0	0,9242	0,9145	176,5	0,9022	0,8899	214,0	0,8806	0,8660	251,5	0,8594	0,8428
139,5	0,9239	0,9141	177,0	0,9019	0,8896	214,5	0,8803	0,8657	252,0	0,8591	0,8425
140,0	0,9236	0,9138	177,5	0,9017	0,8893	215,0	0,8800	0,8654	252,5	0,8588	0,8422
140,5	0,9233	0,9135	178,0	0,9014	0,8890	215,5	0,8797	0,8651	253,0	0,8585	0,8418
141,0	0,9230	0,9131	178,5	0,9011	0,8887	216,0	0,8794	0,8648	253,5	0,8582	0,8415
141,5	0,9227	0,9128	179,0	0,9008	0,8883	216,5	0,8792	0,8645	254,0	0,8580	0,8412
142,0	0,9224	0,9125	179,5	0,9005	0,8880	217,0	0,8789	0,8642	254,5	0,8577	0,8409
142,5	0,9222	0,9122	180,0	0,9002	0,8877	217,5	0,8786	0,8639	255,0	0,8574	0,8406
143,0	0,9219	0,9118	180,5	0,8999	0,8874	218,0	0,8783	0,8635	255,5	0,8571	0,8403
143,5	0,9216	0,9115	181,0	0,8996	0,8871	218,5	0,8780	0,8632	256,0	0,8568	0,8400
144,0	0,9213	0,9112	181,5	0,8993	0,8867	219,0	0,8778	0,8629	256,5	0,8566	0,8397
144,5	0,9210	0,9108	182,0	0,8990	0,8864	219,5	0,8775	0,8626	257,0	0,8563	0,8394
145,0	0,9207	0,9105	182,5	0,8988	0,8861	220,0	0,8772	0,8623	257,5	0,8560	0,8391
145,5	0,9204	0,9102	183,0	0,8985	0,8858	220,5	0,8769	0,8620	258,0	0,8557	0,8388
146,0	0,9201	0,9098	183,5	0,8982	0,8855	221,0	0,8766	0,8617	258,5	0,8554	0,8385
146,5	0,9198	0,9095	184,0	0,8979	0,8851	221,5	0,8763	0,8614	259,0	0,8552	0,8382
147,0	0,9195	0,9092	184,5	0,8976	0,8848	222,0	0,8760	0,8611	259,5	0,8549	0,8379
147,5	0,9192	0,9089	185,0	0,8973	0,8845	222,5	0,8758	0,8608	260,0	0,8546	0,8376
148,0	0,9189	0,9085	185,5	0,8970	0,8842	223,0	0,8755	0,8604	260,5	0,8543	0,8373
148,5	0,9186	0,9082	186,0	0,8967	0,8839	223,5	0,8752	0,8601	261,0	0,8540	0,8370
149,0	0,9183	0,9079	186,5	0,8964	0,8835	224,0	0,8749	0,8598	261,5	0,8538	0,8367
149,5	0,9180	0,9075	187,0	0,8961	0,8832	224,5	0,8746	0,8595	262,0	0,8535	0,8364
150,0	0,9177	0,9072	187,5	0,8959	0,8829	225,0	0,8743	0,8592	262,5	0,8532	0,8361
150,5	0,9174	0,9069	188,0	0,8956	0,8826	225,5	0,8740	0,8589	263,0	0,8529	0,8357
151,0	0,9171	0,9065	188,5	0,8953	0,8823	226,0	0,8737	0,8586	263,5	0,8526	0,8354
151,5	0,9168	0,9062	189,0	0,8950	0,8819	226,5	0,8735	0,8582	264,0	0,8524	0,8351
152,0	0,9165	0,9059	189,5	0,8947	0,8816	227,0	0,8732	0,8579	264,5	0,8521	0,8348
152,5	0,9163	0,9056	190,0	0,8944	0,8813	227,5	0,8729	0,8576	265,0	0,8518	0,8345
153,0	0,9160	0,9052	190,5	0,8941	0,8810	228,0	0,8726	0,8573	265,5	0,8515	0,8342
153,5	0,9157	0,9049	191,0	0,8938	0,8807	228,5	0,8723	0,8570	266,0	0,8512	0,8339
154,0	0,9154	0,9046	191,5	0,8935	0,8803	229,0	0,8721	0,8566	266,5	0,8510	0,8336
154,5	0,9151	0,9042	192,0	0,8932	0,8800	229,5	0,8718	0,8563	267,0	0,8507	0,8333
155,0	0,9148	0,9039	192,5	0,8930	0,8797	230,0	0,8715	0,8560	267,5	0,8504	0,8330
155,5	0,9145	0,9036	193,0	0,8927	0,8794	230,5	0,8712	0,8557	268,0	0,8501	0,8326
156,0	0,9142	0,9033	193,5	0,8924	0,8791	231,0	0,8709	0,8554	268,5	0,8498	0,8323
156,5	0,9139	0,9029	194,0	0,8921	0,8787	231,5	0,8707	0,8551	269,0	0,8496	0,8320
157,0	0,9136	0,9026	194,5	0,8918	0,8784	232,0	0,8704	0,8548	269,5	0,8493	0,8317
157,5	0,9133	0,9023	195,0	0,8915	0,8781	232,5	0,8701	0,8545	270,0	0,8490	0,8314
158,0	0,9130	0,9020	195,5	0,8912	0,8778	233,0	0,8698	0,8541	270,5	0,8487	0,8311
158,5	0,9127	0,9017	196,0	0,8909	0,8775	233,5	0,8695	0,8538	271,0	0,8484	0,8308
159,0	0,9124	0,9013	196,5	0,8906	0,8771	234,0	0,8693	0,8535	271,5	0,8482	0,8305
159,5	0,9121	0,9010	197,0	0,8903	0,8768	234,5	0,8690	0,8532	272,0	0,8479	0,8302
160,0	0,9118	0,9007	197,5	0,8901	0,8765	235,0	0,8687	0,8529	272,5	0,8476	0,8299
160,5	0,9115	0,9004	198,0	0,8898	0,8762	235,5	0,8684	0,8526	273,0	0,8473	0,8296
161,0	0,9112	0,9000	198,5	0,8895	0,8759	236,0	0,8681	0,8523	273,5	0,8470	0,8293
161,5	0,9109	0,8997	199,0	0,8892	0,8755	236,5	0,8678	0,8520	274,0	0,8468	0,8290
162,0	0,9106	0,8994	199,5	0,8889	0,8752	237,0	0,8675	0,8517	274,5	0,8465	0,8287

[C] Use column A factors for asphalts with density at 15°C of 966 kg/m^3 or higher.

[D] Use column B factors for asphalts with density at 15°C of 850 to 965 kg/m^3.

Table 2.9 Temperature-Volume Corrections for Asphalt Materials (Degrees Fahrenheit)

Observed Temperature, °F	Volume Correction Factor to 60°F[A,B]		Observed Temperature, °F	Volume Correction Factor to 60°F[A,B]		Observed Temperature, °F	Volume Correction Factor to 60°F[A,B]		Observed Temperature, °F	Volume Correction Factor to 60°F[A,B]	
	A	B		A	B		A	B		A	B
0	1.0211	1.0241	75	0.9948	0.9940	150	0.9689	0.9647	225	0.9436	0.9361
1	1.0208	1.0237	76	0.9944	0.9936	151	0.9686	0.9643	226	0.9432	0.9358
2	1.0204	1.0233	77	0.9941	0.9932	152	0.9682	0.9639	227	0.9429	0.9354
3	1.0201	1.0229	78	0.9937	0.9929	153	0.9679	0.9635	228	0.9426	0.9350
4	1.0197	1.0225	79	0.9934	0.9925	154	0.9675	0.9632	229	0.9422	0.9346
5	1.0194	1.0221	80	0.9930	0.9921	155	0.9672	0.9628	230	0.9419	0.9343
6	1.0190	1.0217	81	0.9927	0.9917	156	0.9669	0.9624	231	0.9416	0.9339
7	1.0186	1.0213	82	0.9923	0.9913	157	0.9665	0.9620	232	0.9412	0.9335
8	1.0183	1.0209	83	0.9920	0.9909	158	0.9662	0.9616	233	0.9409	0.9331
9	1.0179	1.0205	84	0.9916	0.9905	159	0.9658	0.9612	234	0.9405	0.9328
10	1.0176	1.0201	85	0.9913	0.9901	160	0.9655	0.9609	235	0.9402	0.9324
11	1.0172	1.0197	86	0.9909	0.9897	161	0.9652	0.9605	236	0.9399	0.9320
12	1.0169	1.0193	87	0.9906	0.9893	162	0.9648	0.9601	237	0.9395	0.9316
13	1.0165	1.0189	88	0.9902	0.9889	163	0.9645	0.9597	238	0.9392	0.9313
14	1.0162	1.0185	89	0.9899	0.9885	164	0.9641	0.9593	239	0.9389	0.9309
15	1.0158	1.0181	90	0.9896	0.9881	165	0.9638	0.9589	240	0.9385	0.9305
16	1.0155	1.0177	91	0.9892	0.9877	166	0.9635	0.9585	241	0.9382	0.9301
17	1.0151	1.0173	92	0.9889	0.9873	167	0.9631	0.9582	242	0.9379	0.9298
18	1.0148	1.0168	93	0.9885	0.9869	168	0.9628	0.9578	243	0.9375	0.9294
19	1.0144	1.0164	94	0.9882	0.9865	169	0.9624	0.9574	244	0.9372	0.9290
20	1.0141	1.0160	95	0.9878	0.9861	170	0.9621	0.9570	245	0.9369	0.9286
21	1.0137	1.0156	96	0.9875	0.9857	171	0.9618	0.9566	246	0.9365	0.9283
22	1.0133	1.0152	97	0.9871	0.9854	172	0.9614	0.9562	247	0.9362	0.9279
23	1.0130	1.0148	98	0.9868	0.9850	173	0.9611	0.9559	248	0.9359	0.9275
24	1.0126	1.0144	99	0.9864	0.9846	174	0.9607	0.9555	249	0.9356	0.9272
25	1.0123	1.0140	100	0.9861	0.9842	175	0.9604	0.9551	250	0.9352	0.9268
26	1.0119	1.0136	101	0.9857	0.9838	176	0.9601	0.9547	251	0.9349	0.9264
27	1.0116	1.0132	102	0.9854	0.9834	177	0.9597	0.9543	252	0.9346	0.9260
28	1.0112	1.0128	103	0.9851	0.9830	178	0.9594	0.9539	253	0.9342	0.9257
29	1.0109	1.0124	104	0.9847	0.9826	179	0.9590	0.9536	254	0.9339	0.9253
30	1.0105	1.0120	105	0.9844	0.9822	180	0.9587	0.9532	255	0.9336	0.9249
31	1.0102	1.0116	106	0.9840	0.9818	181	0.9584	0.9528	256	0.9332	0.9245
32	1.0098	1.0112	107	0.9887	0.9814	182	0.9580	0.9524	257	0.9329	0.9242
33	1.0095	1.0108	108	0.9833	0.9810	183	0.9577	0.9520	258	0.9326	0.9238
34	1.0091	1.0104	109	0.9830	0.9806	184	0.9574	0.9517	259	0.9322	0.9234
35	1.0088	1.0100	110	0.9826	0.9803	185	0.9570	0.9513	260	0.9319	0.9231
36	1.0084	1.0096	111	0.9823	0.9799	186	0.9567	0.9509	261	0.9316	0.9227
37	1.0081	1.0092	112	0.9819	0.9795	187	0.9563	0.9505	262	0.9312	0.9223
38	1.0077	1.0088	113	0.9816	0.9791	188	0.9560	0.9501	263	0.9309	0.9219
39	1.0074	1.0084	114	0.9813	0.9787	189	0.9557	0.9498	264	0.9306	0.9216
40	1.0070	1.0080	115	0.9809	0.9783	190	0.9553	0.9494	265	0.9302	0.9212
41	1.0067	1.0076	116	0.9806	0.9779	191	0.9550	0.9490	266	0.9299	0.9208
42	1.0063	1.0072	117	0.9802	0.9775	192	0.9547	0.9486	267	0.9296	0.9205
43	1.0060	1.0068	118	0.9799	0.9771	193	0.9543	0.9482	268	0.9293	0.9201
44	1.0056	1.0064	119	0.9795	0.9767	194	0.9540	0.9478	269	0.9289	0.9197
45	1.0053	1.0060	120	0.9792	0.9763	195	0.9536	0.9475	270	0.9286	0.9194
46	1.0040	1.0056	121	0.9788	0.9760	196	0.9533	0.9471	271	0.9283	0.9190
47	1.0046	1.0052	122	0.9785	0.9756	197	0.9530	0.9467	272	0.9279	0.9186
48	1.0042	1.0048	123	0.9782	0.9752	198	0.9526	0.9463	273	0.9276	0.9182
49	1.0038	1.0044	124	0.9778	0.9748	199	0.9523	0.9460	274	0.9273	0.9179
50	1.0035	1.0040	125	0.9775	0.9744	200	0.9520	0.9456	275	0.9269	0.9175
51	1.0031	1.0036	126	0.9771	0.9740	201	0.9516	0.9452	276	0.9266	0.9171
52	1.0028	1.0032	127	0.9768	0.9736	202	0.9513	0.9448	277	0.9263	0.9168
53	1.0024	1.0028	128	0.9764	0.9732	203	0.9509	0.9444	278	0.9259	0.9164
54	1.0021	1.0024	129	0.9761	0.9728	204	0.9506	0.9441	279	0.9256	0.9160
55	1.0017	1.0020	130	0.9758	0.9725	205	0.9503	0.9437	280	0.9253	0.9157
56	1.0014	1.0016	131	0.9754	0.9721	206	0.9499	0.9433	281	0.9250	0.9153
57	1.0010	1.0012	132	0.9751	0.9717	207	0.9496	0.9429	282	0.9246	0.9149
58	1.0007	1.0008	133	0.9747	0.9713	208	0.9493	0.9425	283	0.9243	0.9146
59	1.0003	1.0004	134	0.9744	0.9709	209	0.9489	0.9422	284	0.9240	0.9142
60	1.0000	1.0000	135	0.9740	0.9705	210	0.9486	0.9418	285	0.9236	0.9138
61	0.9997	0.9996	136	0.9737	0.9701	211	0.9483	0.9414	286	0.9233	0.9135
62	0.9993	0.9992	137	0.9734	0.9697	212	0.9479	0.9410	287	0.9230	0.9131
63	0.9990	0.9988	138	0.9730	0.9693	213	0.9476	0.9407	288	0.9227	0.9127
64	0.9986	0.9984	139	0.9727	0.9690	214	0.9472	0.9403	289	0.9223	0.9124
65	0.9983	0.9980	140	0.9723	0.9686	215	0.9469	0.9399	290	0.9220	0.9120
66	0.9979	0.9976	141	0.9720	0.9682	216	0.9466	0.9395	291	0.9217	0.9116
67	0.9976	0.9972	142	0.9716	0.9678	217	0.9462	0.9391	292	0.9213	0.9113
68	0.9972	0.9968	143	0.9713	0.9674	218	0.9459	0.9388	293	0.9210	0.9109
69	0.9969	0.9964	144	0.9710	0.9670	219	0.9456	0.9384	294	0.9207	0.9105
70	0.9965	0.9960	145	0.9706	0.9666	220	0.9452	0.9380	295	0.9204	0.9102
71	0.9962	0.9956	146	0.9703	0.9662	221	0.9449	0.9376	296	0.9200	0.9098
72	0.9958	0.9952	147	0.9699	0.9659	222	0.9446	0.9373	297	0.9197	0.9094
73	0.9955	0.9948	148	0.9696	0.9655	223	0.9442	0.9369	298	0.9194	0.9091
74	0.9951	0.9944	149	0.9693	0.9651	224	0.9439	0.9365	299	0.9190	0.9087

[A] Use column A factors for asphalts with API gravity at 60°F of 14.9° or less or with specific gravity 60/60°F of 0.967 or higher.

[B] Use column B factors for asphalts with API gravity at 60°F from 15.0° to 34.9° or with specific gravity 60/60°F from 0.850 to 0.966.

Table 2.9 (Continued) Temperature-Volume Corrections for Asphalt Materials (Degrees Fahrenheit)

Observed Temperature, °F	Volume Correction Factor to 60°F[A,#]		Observed Temperature, °F	Volume Correction Factor to 60°F[A,#]		Observed Temperature, °F	Volume Correction Factor to 60°F[A,#]		Observed Temperature, °F	Volume Correction Factor to 60°F[A,#]	
	A	B		A	B		A	B		A	B
300	0.9187	0.9083	350	0.9024	0.8902	400	0.8864	0.8724	450	0.8705	0.8550
301	0.9184	0.9080	351	0.9021	0.8899	401	0.8861	0.8721	451	0.8702	0.8547
302	0.9181	0.9076	352	0.9018	0.8895	402	0.8857	0.8717	452	0.8699	0.8543
303	0.9177	0.9072	353	0.9015	0.8891	403	0.8854	0.8714	453	0.8696	0.8540
304	0.9174	0.9069	354	0.9011	0.8888	404	0.8851	0.8710	454	0.8693	0.8536
305	0.9171	0.9065	355	0.9008	0.8884	405	0.8848	0.8707	455	0.8690	0.8533
306	0.9167	0.9061	356	0.9005	0.8881	406	0.8845	0.8703	456	0.8687	0.8529
307	0.9164	0.9058	357	0.9002	0.8877	407	0.8841	0.8700	457	0.8683	0.8526
308	0.9161	0.9054	358	0.8998	0.8873	408	0.8838	0.8696	458	0.8680	0.8522
309	0.9158	0.9050	359	0.8995	0.8870	409	0.8835	0.8693	459	0.8677	0.8519
310	0.9154	0.9047	360	0.8992	0.8866	410	0.8832	0.8689	460	0.8674	0.8516
311	0.9151	0.9043	361	0.8989	0.8863	411	0.8829	0.8686	461	0.8671	0.8512
312	0.9148	0.9039	362	0.8986	0.8859	412	0.8826	0.8682	462	0.8668	0.8509
313	0.9145	0.9036	363	0.8982	0.8856	413	0.8822	0.8679	463	0.8665	0.8505
314	0.9141	0.9032	364	0.8979	0.8852	414	0.8819	0.8675	464	0.8661	0.8502
315	0.9138	0.9029	365	0.8976	0.8848	415	0.8816	0.8672	465	0.8658	0.8498
316	0.9135	0.9025	366	0.8973	0.8845	416	0.8813	0.8668	466	0.8655	0.8495
317	0.9132	0.9021	367	0.8969	0.8841	417	0.8810	0.8665	467	0.8652	0.8492
318	0.9128	0.9018	368	0.8966	0.8838	418	0.8806	0.8661	468	0.8649	0.8488
319	0.9125	0.9014	369	0.8963	0.8834	419	0.8803	0.8658	469	0.8646	0.8485
320	0.9122	0.9010	370	0.8960	0.8831	420	0.8800	0.8654	470	0.8643	0.8481
321	0.9118	0.9007	371	0.8957	0.8827	421	0.8797	0.8651	471	0.8640	0.8478
322	0.9115	0.9003	372	0.8953	0.8823	422	0.8794	0.8647	472	0.8636	0.8474
323	0.9112	0.9000	373	0.8950	0.8820	423	0.8791	0.8644	473	0.8633	0.8471
324	0.9109	0.8996	374	0.8947	0.8816	424	0.8787	0.8640	474	0.8630	0.8468
325	0.9105	0.8992	375	0.8944	0.8813	425	0.8784	0.8637	475	0.8627	0.8464
326	0.9102	0.8989	376	0.8941	0.8809	426	0.8781	0.8633	476	0.8624	0.8461
327	0.9099	0.8985	377	0.8937	0.8806	427	0.8778	0.8630	477	0.8621	0.8457
328	0.9096	0.8981	378	0.8934	0.8802	428	0.8775	0.8626	478	0.8618	0.8454
329	0.9092	0.8978	379	0.8931	0.8799	429	0.8772	0.8623	479	0.8615	0.8451
330	0.9089	0.8974	380	0.8928	0.8795	430	0.8768	0.8619	480	0.8611	0.8447
331	0.9086	0.8971	381	0.8924	0.8792	431	0.8765	0.8616	481	0.8608	0.8444
332	0.9083	0.8967	382	0.8921	0.8788	432	0.8762	0.8612	482	0.8605	0.8440
333	0.9079	0.8963	383	0.8918	0.8784	433	0.8759	0.8609	483	0.8602	0.8437
334	0.9076	0.8960	384	0.8915	0.8781	434	0.8756	0.8605	484	0.8599	0.8433
335	0.9073	0.8956	385	0.8912	0.8777	435	0.8753	0.8602	485	0.8596	0.8430
336	0.9070	0.8952	386	0.8908	0.8774	436	0.8749	0.8599	486	0.8593	0.8427
337	0.9066	0.8949	387	0.8905	0.8770	437	0.8746	0.8595	487	0.8590	0.8423
338	0.9063	0.8945	388	0.8902	0.8767	438	0.8743	0.8592	488	0.8587	0.8420
339	0.9060	0.8942	389	0.8899	0.9763	439	0.8740	0.8588	489	0.8583	0.8416
340	0.9057	0.8938	390	0.8896	0.8760	440	0.8737	0.8585	490	0.8580	0.8413
341	0.9053	0.8934	391	0.8892	0.8756	441	0.8734	0.8581	491	0.8577	0.8410
342	0.9050	0.8931	392	0.8889	0.8753	442	0.8731	0.8578	492	0.8574	0.8406
343	0.9047	0.8927	393	0.8886	0.8749	443	0.8727	0.8574	493	0.8571	0.8403
344	0.9044	0.8924	394	0.8883	0.8746	444	0.8724	0.8571	494	0.8568	0.8399
345	0.9040	0.8920	395	0.8880	0.8742	445	0.8721	0.8567	495	0.8565	0.8396
346	0.9037	0.8916	396	0.8876	0.8738	446	0.8718	0.8564	496	0.8562	0.8393
347	0.9034	0.8913	397	0.8873	0.8735	447	0.8715	0.8560	497	0.8559	0.8389
348	0.9031	0.8909	398	0.8870	0.8731	448	0.8712	0.8557	498	0.8556	0.8386
349	0.9028	0.8906	399	0.8867	0.8728	449	0.8709	0.8554	499	0.8552	0.8383
									500	0.8549	0.8379

[A] Use column A factors for asphalts with API gravity at 60°F of 14.9° or less or with specific gravity 60/60°F of 0.967 or higher.

[#] Use column B factors for asphalts with API gravity at 60°F from 15.0° to 34.9° or with specific gravity 60/60°F from 0.850 to 0.966.

Table 2.10 Temperature-Volume Corrections for Emulsified Asphalts

Legend: t = observed temperature in degrees Fahrenheit (Celsius)
M = multiplier for correcting volumes to the basis of 15.6°C (60°F)

t		M*	t		M*	t		M*
°C	°F		°C	°F		°C	°F	
10.0	**50**	1.00250	35.0	**95**	.99125	57.8	**136**	.98100
10.6	**51**	1.00225	35.6	**96**	.99100	58.3	**137**	.98075
11.1	**52**	1.00200	36.1	**97**	.99075	58.9	**138**	.98050
11.7	**53**	1.00175	36.7	**98**	.99050	59.4	**139**	.98025
12.2	**54**	1.00150	37.2	**99**	.99025	60.0	**140**	.98000
12.8	**55**	1.00125	37.8	**100**	.99000	60.6	**141**	.97975
13.3	**56**	1.00100	38.3	**101**	.98975	61.1	**142**	.97950
13.9	**57**	1.00075	38.9	**102**	.98950	61.7	**143**	.97925
14.4	**58**	1.00050	39.4	**103**	.98925	62.2	**144**	.97900
15.0	**59**	1.00025	40.0	**104**	.98900	62.8	**145**	.97875
15.6	**60**	1.00000	40.6	**105**	.98875	63.3	**146**	.97850
16.1	**61**	.99975	41.1	**106**	.98850	63.9	**147**	.97825
16.7	**62**	.99950	41.7	**107**	.98825	64.4	**148**	.97800
17.2	**63**	.99925	42.2	**108**	.98800	65.0	**149**	.97775
17.8	**64**	.99900	42.8	**109**	.98775	65.6	**150**	.97750
18.3	**65**	.99875	43.3	**110**	.98750	66.1	**151**	.97725
18.9	**66**	.99850	43.9	**111**	.98725	66.7	**152**	.97700
19.4	**67**	.99825	44.4	**112**	.98700	67.2	**153**	.97675
20.0	**68**	.99800	45.0	**113**	.98675	67.8	**154**	.97650
20.6	**69**	.99775	45.6	**114**	.98650	68.3	**155**	.97625
21.1	**70**	.99750	46.1	**115**	.98625	68.9	**156**	.97600
21.7	**71**	.99725	46.7	**116**	.98600	69.4	**157**	.97575
22.2	**72**	.99700	47.2	**117**	.98575	70.0	**158**	.97550
22.8	**73**	.99675	47.8	**118**	.98550	70.6	**159**	.97525
23.3	**74**	.99650	48.3	**119**	.98525	71.1	**160**	.97500
23.9	**75**	.99625	48.9	**120**	.98500	71.7	**161**	.97475
24.4	**76**	.99600	49.4	**121**	.98475	72.2	**162**	.97450
25.0	**77**	.99575	50.0	**122**	.98450	72.8	**163**	.97425
25.6	**78**	.99550	50.6	**123**	.98425	73.3	**164**	.97400
26.1	**79**	.99525	51.1	**124**	.98400	73.9	**165**	.97375
26.7	**80**	.99500	51.7	**125**	.98375	74.4	**166**	.97350
27.2	**81**	.99475	52.2	**126**	.98350	75.0	**167**	.97325
27.8	**82**	.99450	52.8	**127**	.98325	75.6	**168**	.97300
28.3	**83**	.99425	53.3	**128**	.98300	76.1	**169**	.97275
28.9	**84**	.99400	53.9	**129**	.98275	76.7	**170**	.97250
29.4	**85**	.99375	54.4	**130**	.98250	77.2	**171**	.97225
30.0	**86**	.99350	55.0	**131**	.98225	77.8	**172**	.97200
30.6	**87**	.99325	55.6	**132**	.98200	78.3	**173**	.97175
31.1	**88**	.99300	56.1	**133**	.98175	78.9	**174**	.97150
31.7	**89**	.99275	56.7	**134**	.98150	79.4	**175**	.97125
32.2	**90**	.99250	57.2	**135**	.98125			
32.8	**91**	.99225						
33.3	**92**	.99200						
33.9	**93**	.99175						
34.4	**94**	.99150						

After the specific gravity of asphalt is found at 25/25°C (77/77°F), its specific gravity at 15.6/15.6°C (60/60°F) can be calculated by expressing Eq. 2b for both temperatures, combining, appropriately substituting Eq. 1 and simplifying to become

$$G_{15.6/15.6} = \frac{G_{25/25} \ \gamma_{w25}}{M_{25} \ \gamma_{w15.6}} \quad \text{or} \quad \left(G_{60/60} = \frac{G_{77/77} \ \gamma_{w77}}{M_{77} \ \gamma_{w60}} \right) \tag{3}$$

where γ_{w25} (γ_{77}) = density of water at 25°C (77°F) = 0.9970g/ml,
$\gamma_{w15.6}$ (γ_{w60}) = density of water at 15.6°C (60°F) = 0.9988g/ml,
M_{25} (M_{77}) = multiplier from Table 2.8, 2.9, or 2.10

Assume the specific gravity of an asphalt cement at 77/77° F is 1.003. From Table 2.9, M for 77°F is 0.9941 (Column A). The specific gravity at 60/60°F is then

$$\frac{1.003\,(0.9970)}{0.9941\,(0.9988)} = 1.007$$

Horizontal Tank Measurements

Many asphalt containers for transporting and storage are cylindrical tanks in a horizontal position. Measurements of the volume of material in the tank consists of measuring its depth. Table 2.11 gives quantities in terms of percent of the total capacity based on the depth in percent of the diameter.

Volumes of asphalt are usually expressed in litres (gallons), and the formula for determining the capacity of a cylindrical tank is

$$V = 785\,D^2\,L \qquad (V = 5.88\,D^2L) \tag{4}$$

where V = volumes, litres (gal);
D = diameter of interior of tank, m (ft.); and
L = length of interior of tank, m (ft.)

Example: (U.S. Customary Units)

Assume a horizontal tank has a capacity of 12,740 gal and a diameter of 9 ft 6 in. If the depth of the asphalt in the tank is 7 ft 3½ in. the percent depth filled is

$$\frac{7.29\,(100)}{9.5} = 76.8 \text{ percent}$$

By interpolating in Table 2.11, the volume of asphalt in the tank is

$$\frac{12{,}740\,(82.40)}{100} = 10{,}498 \text{ gal}$$

When the tank is more than half full, as in the preceding example, it may be more desirable to determine the volume of the unfilled space and deduct this from the capacity. Using the above example, the computations would be:

$$12{,}740 - 12{,}740\ \frac{(100\text{-}82.40)}{100} = 12{,}740 - 2{,}242 = 10{,}498 \text{ gal}$$

Table 2.11 Percent Capacities for Various Depths of Cylindrical Tanks in Horizontal Position

Percent Depth Filled	Percent of Capacity	Percent Depth Filled	Percent of Capacity	Percent Depth Filled	Percent of Capacity	Percent Depth Filled	Percent of Capacity
1	0.20	26	20.73	51	51.27	76	81.50
2	0.50	27	21.86	52	52.55	77	82.60
3	0.90	28	23.00	53	53.81	78	83.68
4	1.34	29	24.07	54	55.08	79	84.74
5	1.87	30	25.31	55	56.34	80	85.77
6	2.45	31	26.48	56	57.60	81	86.77
7	3.07	32	27.66	57	58.86	82	87.76
8	3.74	33	28.84	58	60.11	83	88.73
9	4.45	34	30.03	59	61.36	84	89.68
10	5.20	35	31.19	60	62.61	85	90.60
11	5.98	36	32.44	61	63.86	86	91.50
12	6.80	37	33.66	62	65.10	87	92.36
13	7.64	38	34.90	63	66.34	88	93.20
14	8.50	39	36.14	64	67.56	89	94.02
15	9.40	40	37.39	65	68.81	90	94.80
16	10.32	41	38.64	66	69.97	91	95.55
17	11.27	42	39.89	67	71.16	92	96.26
18	12.24	43	41.14	68	72.34	93	96.93
19	13.23	44	42.40	69	73.52	94	97.55
20	14.23	45	43.66	70	74.69	95	98.13
21	15.26	46	44.92	71	75.93	96	98.66
22	16.32	47	46.19	72	77.00	97	99.10
23	17.40	48	47.45	73	78.14	98	99.50
24	18.50	49	48.73	74	79.27	99	99.80
25	19.61	50	50.00	75	80.39		

2.6 Safety Procedures for Hot Asphalt

Since asphalt products are often stored and handled at elevated temperatures, fire prevention is extremely important. One of the greatest hazards in handling hot asphalt is exposure to a source of ignition. Sources of ignition, such as sparks of electrical or other origin, open flames or incandescent material (lighted cigarette) should be prohibited or otherwise strictly controlled in the vicinity of asphalt operations.

Many of the metal surfaces in asphalt plants and in roofing or processing operations exceed 65° C (150° F), and frequently are well over 93° C (200° F). Asphalt temperatures commonly exceed 149° C (300° F) as used in these areas. Exposed lines or surfaces can burn flesh on momentary contact—65° C (150° F) or higher. Also, similar hazards exist in other areas; around dryers, boiler houses, asphalt receiving and handling areas, and similar locations.

Therefore:

- Be aware of burn hazards in the workplace.
- Burn hazards should be clearly indicated by warning signs.
- Wear appropriate protective clothing.
- Follow proper safety procedures. Use catwalks, screens, barrier guards and shields to protect from steam, hot asphalt, hot surfaces, and similar danger.

Never enter an empty asphalt tank unless it is properly ventilated and has been purged of asphalt fumes. It is advisable to have a second person in the immediate proximity as an observer to be assured that one does not lose consciousness due to lack of oxygen.

Whenever a person is injured from exposure to asphalt fumes or hot asphalt, obtain first aid / medical attention immediately. If hot asphalt comes in contact with the skin, the affected area should be immersed in cool water. Do not attempt to wipe off the hot asphalt with a cloth as this may remove skin along with the asphalt. To prevent the possibility of future medical complications, have the victim examined by a physician even if the injury does not appear to be serious.

From a safety standpoint, it is desirable to store asphalt at a temperature as far as possible below the flash point. However, it should be recognized that the values reported by the flash point tests are specific to the test procedures employed and not necessarily representative of the vapor space atmospheres existent in storage.

Table 2.12 contains the recommended storage temperatures for various types and grades of asphalt. The guidelines are not intended to establish a

Table 2.12 **Guideline Temperatures for Storage and Handling of Asphalt Products**

Type & Grade	Spec. Ref.	Min. Flash °C (°F)	Storage Temperature °C (°F)
AC-2.5	AASHTO M226	163 (325)	160 (320)
AC-5	ASTM D 3381	177 (350)	166 (330)
AC-10		219 (425)	174 (345)
AC-20		232 (450)	177 (350)
AC-40		232 (450)	177 (350)
AR-1000	AASHTO M226	205 (400)	163 (325)
AR-2000	ASTM D 3381	219 (425)	168 (335)
AR-4000		227 (440)	177 (350)
AR-8000		232 (450)	177 (350)
AR-16000		238 (460)	177 (350)
Pen 40-50	AASHTO M20	232 (450)	177 (350)
Pen 60-70	ASTM D 946	232 (450)	177 (350)
Pen 85-100		232 (450)	177 (350)
Pen 120-150		219 (425)	177 (350)
Pen 200-300		177 (350)	168 (335)
MC-30	AASHTO M82	38 (100)	54 (130)
MC-70	ASTM D 2027	38 (100)	71 (160)
MC-250		66 (150)	91 (195)
MC-800		66 (150)	99 (210)
MC-3000		66 (150)	99 (210)
RC-70	AASHTO M81	— —	71 (160)
RC-250	ASTM D 2028	27 (80)	91 (195)
RC-800		27 (80)	99 (210)
RC-3000		27 (80)	99 (210)
SC-70		66 (150)	71 (160)
SC-250	ASTM D 2026	79 (175)	91 (195)
SC-800		93 (200)	99 (210)
SC-3000		107 (225)	99 (210)
All Grades of Emulsified Asphalt	AASHTO M140 & 208 ASTM D 977 & 2397	— —	82 (180)
Dampproofing A	AASHTO M115	175 (350)	182 (360)
B	ASTM D 449	205 (400)	210 (410)
C		205 (400)	218 (450)
Primer	AASHTO M116	— —	54 (130)
	ASTM D 41	— —	54 (130)
Underseal	AASHTO M238	218 (425)	232 (450)
	ASTM D 3141		
Membrane	AASHTO M239	218 (425)	232 (450)
	ASTM D 2521		
B-U Roofing I	ASTM D 312	225 (437)	191 (375)
II		225 (437)	218 (425)
III		225 (437)	232 (450)
IV		225 (437)	246 (475)
Residual Fluxes	Industry Sources	260 (500)	177 (350)
		288 (550)	191 (375)

rigid standard, but to indicate temperatures for good practice that also allow for practical product storage, needs of pumpability, and compliance with applicable environmental regulations.

Storage temperatures in the 100-130° C (212-265° F) range are particularly troublesome. Storage temperature should be either below 100° C (212° F) or above 130° C (265° F). In particular, temperatures should not be allowed to cycle above and below the boiling point of water.

Chapter 3

Mineral Aggregates

This chapter concentrates on aggregates, which make up the major portion (90-95% by weight) of any asphalt paving mixture. Familiarity with aggregate materials will aid in the understanding of the asphalt-aggregate relationship. A description of the characteristics and properties of aggregate materials and their sources also is helpful in understanding the design, construction and performance of paving mixtures.

The methods of proportioning aggregates described in this chapter are designed to provide a means of obtaining successful blends, with a minimum of repetitive trial solutions.

SECTION 3.1 Aggregates for Asphalt Mixtures
☐ Introduction
☐ Origin of Rocks
☐ Sources of Aggregates
☐ Aggregate Properties

SECTION 3.2 Aggregate Analysis
☐ Gradation
☐ Specific Gravity
☐ Surface Area

SECTION 3.3 Gradation Blending Calculations
☐ Introduction
☐ Proportioning Determinations
☐ Grading Adjustments

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3.1 Aggregates for Asphalt Mixtures

Introduction

The amount of mineral aggregate in asphalt paving mixtures is generally 90 to 95 percent by weight or 75 to 85 percent by volume. Mineral aggregate is primarily responsible for the load supporting capacity of pavement; accordingly, asphalt pavement performance is heavily influenced by aggregate.

Mineral aggregate has been defined as any hard, inert mineral material used for mixing in graduated particles or fragments. It includes sand, gravel, crushed stone, slag, rock dust or powder.

There are other definitions of mineral aggregate that generally reflect the purpose for which the material is intended. For example, ASTM Designation D 8 defines aggregate as "a granular material of mineral composition such as sand, gravel, shell, slag, or crushed stone, used with a cementing medium to form mortars or concrete or alone as in base courses, railroad ballasts, etc."

Origin of Rocks

It is not the purpose of this Section to describe in detail the origin of bedrock, from which all aggregate materials are derived. A brief discussion is in order, however, since knowledge of the origin of rock is important for understanding the production of mineral aggregates.

All rocks are divided into three general classifications: sedimentary, igneous, and metamorphic (Figure 3.1). These classifications indicate the means by which the rocks were formed.

Sedimentary Rocks

Sedimentary rocks are formed by the accumulation of sediment in water or deposited by wind. The sediment may consist of rock or mineral fragments or particles of various sizes (conglomerate, sandstone, shale); or the remains or products of animals or plants (certain limestones and coal); or the product of chemical action or evaporation (salt, gypsum); or of mixtures of these materials. Some sedimentary deposits consist of particles blown from volcanoes and deposited on land or in water. A characteristic feature of sedimentary deposits is a layered structure. This stratification is a direct result of the method of formation; the material having been deposited over some lake or sea bottom.

Class	Type	Family
Sedimentary	Calcareous	Limestone
		Dolomite
	Siliceous	Shale
		Sandstone
		Chert
		Conglomerate[1]
		Breccia[1]
Igneous	Intrusive (coarse-grained)	Granite[2]
		Syenite[2]
		Diorite[2]
		Gabbro
		Periodotite
		Pyroxenite
		Hornblendite
	Extrusive (fine-grained)	Obsidian
		Pumice
		Tuff
		Ryholite[2,3]
		Trachyte[2,3]
		Andesite,[2,3]
		Basalt[2]
		Diabase
Metamorphic	Foliated	Gneiss
		Schist
		Amphibolite
		Slate
	Nonfoliated	Quartzite
		Marble
		Serpentinite

[1] May also be composed partially or entirely of calcareous materials.

[2] Frequently occurs as a porphyritic rock.

[3] Included in general term "felsite" when constituent minerals cannot be determined quantitatively.

Figure 3.1 Different Aggregates Classifications.

Igneous Rocks

Igneous rocks are formed by the cooling and solidification of hot molten materials (magma) erupted from or trapped beneath the earth's crust. They are divided into extrusive and intrusive rocks. Extrusive rocks are formed by material pouring out on the earth's surface, such as that produced by volcanic action, and are distinguished in general by their glass-like structure (rhyolite, andesite, basalt, etc.). Intrusive rocks are formed by cooling and solidification at great depths within the earth's crust. They are wholly crystalline in texture (granite, diorite, gabbro, etc.). While originally formed beneath the earth's surface, they are often exposed today as a result of earth movements and the erosion processes.

Metamorphic Rocks

Metamorphic rocks are generally formed by the modification of sedimentary and igneous rocks as a result of intense pressures set up by severe earth movements and by excessive heat and solutions. Factors causing such modification are complex, and the original form of the altered rock is often difficult to determine. One distinguishing feature of metamorphic rocks is the parallel planes in which the minerals that form rocks are arranged. This arrangement is called foliation. These planes are zones of weakness along which the rocks may be split much more easily than in other directions. Examples of foliated rocks are gneisses and schists (altered from igneous material), and slate (altered from shale). Marble (altered from limestone) and quartzite (altered from sandstone) are common types of metamorphic rocks having a massive texture, usually without foliation.

Sources of Aggregates

Aggregates for asphalt paving are generally classified according to their source of means of preparation. They include pit or bank-run aggregates, processed aggregates, and synthetic or artificial aggregates.

Pit or Bank-Run Materials

Gravel and sand are natural aggregates and are typically pit or bank-run material. Gravel is usually defined as aggregate particles ranging in size from 75 mm (3 in.) to 4.75 mm (No. 4). Sand consists of particles ranging in size from 4.75 mm (No. 4) to 75 μm (No. 200). Sizes 75 μm (No. 200) and smaller include the silt fraction.

Exposed rocks are eroded and degraded by many processes of nature, both physical and chemical. The products of the degradation processes are usually moved by wind, water, or moving ice, and deposited as a soil mineral in various land forms.

Gravels are widely distributed but deposits are rarely found without some proportion of sand and possibly silt. Sandy soil mixtures frequently have some clay and silt. Beach sands, some of which are now far inland, are a uniform sized material, but river sands often contain large amounts of gravel, silt, and clay.

Transporting agents also influence the shape of the particles. Rounded boulders and pebbles included in deposits in glaciated areas are one example. Another is the smooth rounded gravel and sand particles in flowing streams.

Gravels and sand deposits usually are screened to proper size and washed to remove dirt before being used for asphalt paving purposes.

Processed Aggregates

Processed aggregate includes natural gravel or stone that has been crushed and screened. Natural gravel is sometimes crushed to make it more suitable for use in asphalt paving mixtures. Crushing typically improves the particle shape by making the rounded particles more angular in shape and improves the distribution and range of particle sizes.

Crushed stone results from crushing fragments of bedrock or large stones, with all the aggregate particles having fractured faces. In the manufacture of crushed stone, solid ledges of bedrock are broken up in a quarry by blasting and further reduced in size by rock crushers. The crushed product may then be screened to produce desired sizes of aggregate. Sometimes, for reasons of economy, the crushed product is used as it comes from the crusher with little or no screening. This is referred to as "crusher-run" aggregate, and in many instances can be used satisfactorily in asphalt pavement contruction, especially if the graduation of particle sizes is reasonably distributed.

In the processing of crushed limestone, the rock dust produced in the crushing operation is usually separated from other crushed material of 6 mm (¼ in.) or more diameter. This material may either be used as a crushed sand aggregate or processed further to maximum size of 0.6 mm (0.025 in.) in diameter for use as mineral filler in asphalt paving mixtures.

Synthetic or Artificial Aggregates

Aggregates resulting from the modification of materials, which may involve both physical and chemical changes, are sometimes called synthetic or artificial aggregates. They may take the form of the by-product that is developed in the refining of ore, or those specially produced or processed from raw materials for ultimate use as aggregate.

Blast-furnace slag is the most commonly used artificial aggregate. It is the by-product of the smelting of iron in blast furnaces. It is nonmetallic and floats on molten iron. It is drawn off at intervals and reduced in size either by quenching in water or by crushing it after it has air-cooled.

Manufactured aggregates are relatively new in asphalt paving. Typically, they are lightweight and have unusual resistance to wear. Their use is often preferred in bridge-deck or roof-deck paving and in surface layers of asphalt pavements where a high degree of skid resistance is required. These aggregates are manufactured by a firing process and are usually made from clay, shale, slate, processed diatomaceous earth, volcanic glasses, expanded slag, and other like materials. They are produced and marketed under a variety of trade names.

Aggregate Properties

Selecting an aggregate material for use in an asphalt pavement depends upon the availability, cost, and quality of the material, as well as the type of construction that is intended. The suitability of aggregates for use in asphalt construction is determined by evaluating the material in terms of the following:

1. Size and grading
2. Cleanliness
3. Toughness
4. Surface texture
5. Particle shape
6. Absorption
7. Affinity for asphalt.

Size and Grading

The maximum size of an aggregate designates the smallest sieve through which 100 percent of the material will pass. The nominal maximum size is the largest sieve size upon which any of the aggregate material is (or is permitted to be) retained.

Maximum size and aggregate grading are invariably controlled by specifications which prescribe the distribution of particle sizes to be used for a particular aggregate material. The minimum lift thickness of a hot-mix asphalt course is directly related to the maximum aggregate size. Figure 3.2 shows a typical specification controlling the size and grading of an aggregate for an asphalt-concrete surface course. Aggregates are sometimes described on the basis of their gradation. Some examples are (a) dense-graded, (b) open-graded, (c) one-sized, (d) coarse-graded, (e) fine-graded, (f) gap-graded.

Cleanliness

Some aggregates contain certain foreign or deleterious substances that make them undesirable for asphalt paving mixtures unless the amount of foreign matter is reduced. Specifications for such aggregates normally contain a section in which deleterious materials are named, and a limit is placed on the amount of each that is allowable in the aggregate. Typical objectionable materials are vegetation, shale, soft particles, clay lumps, and clay-coating on coarse-aggregate particles.

Aggregate cleanliness may often be determined by visual inspection, but a washed-sieve analysis generally provides positive proof. The sand-equivalent test, developed by the California Division of Highways and described in ASTM D 2419 (AASHTO T 176) is a method of determining the relative proportion of detrimental fine dust or clay-like materials in the portion of the aggregate passing the 4.75 mm (No. 4) sieve.

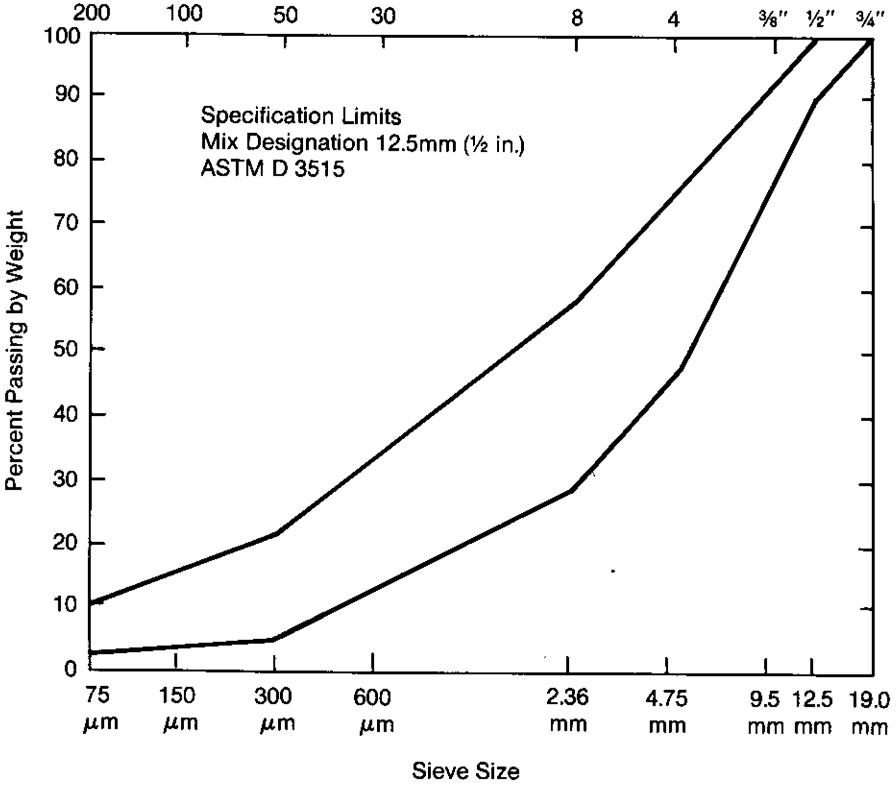


Figure 3.2 **Gradation Curve of an Aggregate Blend for an Asphalt Concrete Surface Course.**

Toughness

Aggregates are subject to additional crushing and abrasive wear during manufacture, placing, and compaction of asphalt paving mixes. Aggregates are also subject to abrasion under traffic loads. They must exhibit, to a certain degree, an ability to resist crushing, degradation, and disintegration. Aggregates at or near the pavement surface require greater toughness than aggregate in the lower layers where loads have dissipated or are not as concentrated.

The Los Angeles Abrasion test measures wear or abrasion resistance of mineral aggregate. Equipment and procedures for this test are detailed in AASHTO T 96 and ASTM C 131. The abrasion machine is illustrated in Figure 3.3

Relatively high resistance to wear, as indicated by a low percent of abrasion loss, is a desirable characteristic of aggregates to be used in asphalt pavement surface layers. Aggregates having higher abrasion losses, within limits, may generally be used in lower pavement layers where they will not be subject to the high stresses caused by traffic.

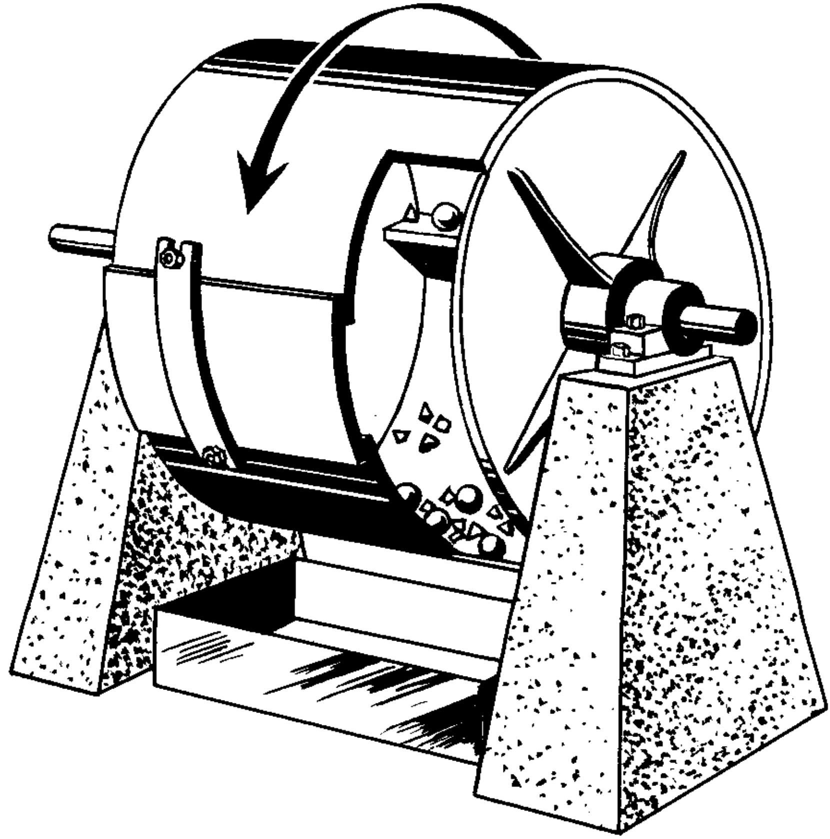


Figure 3.3 Los Angeles Abrasion Machine.

Surface Texture

Like particle shape, the surface texture also influences the workability and strength of asphalt paving mixtures. Surface texture has often been considered more important than the shape of the aggregate particles. A rough, sandpaper-like surface texture as opposed to a smooth surface tends to increase the strength of the paving mix and requires additional asphalt to overcome the loss of workability. Voids in the compacted mineral aggregate are also usually greater, which provide additional space for the required increase of asphalt.

Natural gravels, such as river gravels, often have a smooth surface texture as well as round-shaped particles. Crushing, however, often produces a rough surface texture (especially along the fractured face) and changes the particle shape. Smooth-surfaced aggregates may be easier to coat with an asphalt film but the film will more readily adhere to rough surfaces.

There is not an established method for measuring surface texture but, like particle shape, this characteristic is reflected in strength tests and in the workability of many asphalt mixes.

Particle Shape

Particle shape changes the workability of the paving mix as well as the compactive effort necessary to obtain the required density. Particle shape also has an effect on the strength of the asphalt mix. Irregular or angular particles, such as crushed stone and gravels, and some natural gravels and sands, tend to interlock when compacted and resist displacement. Best interlock is generally obtained with sharp-cornered cubical-shaped particles; it is generally least for round particles.

Rounded particles, such as most natural gravels and sands from stream beds, are used successfully in asphalt paving mixes. However, since a higher density is possible with round aggregate particles, the amount of asphalt in the mix is usually a more critical factor in dense-graded mixes.

Many asphalt mixes contain both angular and rounded aggregate particles. The coarse aggregate fraction is usually a crushed stone or crushed gravel, and the fine aggregate is usually a natural sand (rounded particles) or stone screenings. Such mixes rely principally on crushed aggregate for strength and rounded sand particles for workability and ease of compaction.

A hot-mix asphalt can generally be made more stable, and resistant to rutting by requiring that a significant portion (i.e. 50%) of the coarse aggregate be crushed/angular.

Absorption

The porosity of an aggregate is generally indicated by the amount of water it absorbs when soaked in water. A porous aggregate will also absorb asphalt which will tend to make an asphalt mixture dry or less cohesive.

An extra amount of asphalt must be incorporated into the paving mix to compensate for the absorption of asphalt by the aggregate. Moreover, aggregates that are very porous tend to require a significant amount of extra asphalt to make up for the high absorption rate. Highly porous aggregates normally are not used unless they possess certain other qualities or properties that make them desirable in spite of the high absorption rate. Blast furnace slag and many synthetic or manufactured aggregates are lightweight materials that are also highly porous. Their lightness and wear-resistant properties frequently outweigh the high-absorption rate consideration for use in pavement construction.

Affinity for Asphalt

Stripping—separation of the asphalt film from the aggregate through the action of water—may make an aggregate material unsuitable for asphalt paving mixes. Such material is referred to as hydrophilic (water loving). Siliceous aggregates such as quartzite and some granites are examples of aggregates that may require attention from the stripping viewpoint.

Aggregates that exhibit a high degree of resistance to film stripping in the presence of water are usually suitable for asphalt paving mixes. Such aggregates are referred to as hydrophobic (water hating) aggregates.

Limestone, dolomite, and traprock are usually highly resistant to asphalt film stripping.

Why hydrophobic or hydrophilic aggregates behave as they do is not completely understood. The explanation is not so important as the ability to detect the properties and avoid use of aggregates conducive to asphalt stripping.

A test method for determining the coating and stripping of asphalt-aggregate mixes is described in ASTM D 1664. The uncompacted mixture is soaked in water and the coated particles are evaluated visually. Another test, commonly known as the immersion-compression test, is detailed in ASTM D 1075 and AASHTO T 165. The strength of a compacted asphalt paving mixture, after having been soaked in water, is compared to the strength of an identical specimen that was not soaked. The reduction in strength is an indication of the quality of the aggregate from the standpoint of its resistance to stripping in water. The required percent retained strength is normally set by the user agency.

Other test methods include the Lottman procedure AASHTO T 283, the Root-Tunnicliff procedure as described in National Cooperative Highway Research Program Report 274 and a quick field boil test, ASTM D 3625.

When unsuitable or questionable aggregates must be used, they may often be used satisfactorily if a desirable density-voids relationship can be achieved by adjusting the gradation and asphalt cement. The gradation of the questionable material can be adjusted by blending with other aggregates. Then, by selecting the proper asphalt content to decrease the voids, the compacted pavement will be more impermeable. Such pavements will be more resistant to detrimental effects of water.

A number of liquid antistrip additives are available to improve water resistance. In addition, hydrated lime can be used for this purpose.

Additional information may be found in *Cause and Prevention of Stripping in Asphalt Pavements,* ES-10, Asphalt Institute.

3.2 Aggregate Analysis

Gradation

Aggregate gradation is the distribution of particle sizes expressed as a percent of the total weight. Gradation is determined by passing the material through a series of sieves stacked with progressively smaller openings, and weighing the material retained on each sieve (Figure 3.4). Sieve sizes most often used in grading aggregates for asphalt paving mixes are given in Table 3.1, but not all sizes are necessarily used in each gradation specification.

Table 3.1 Nominal Dimensions of U.S. Standard Sieves

Sieve designation		Sieve opening	
Standard	Alternative	mm	in.
38.1 mm	1½-in.	38.1	1.50
25.0 mm	1-in.	25.0	1.00
19.0 mm	¾-in.	19.0	0.750
12.5 mm	½-in.	12.5	0.500
9.5 mm	⅜-in.	9.5	0.375
4.75 mm	No. 4	4.75	0.187
2.36 mm	No. 8	2.36	0.0937
1.18 mm	No. 16	1.18	0.0469
600 μm	No. 30	0.600	0.0234
300 μm	No. 50	0.300	0.0117
150 μm	No. 100	0.150	0.0059
75 μm	No. 200	0.075	0.0029

Aggregate grading specifications have been developed because of the need (a) to control construction materials and in turn to obtain a desirable and quality pavement; (b) to obtain optimum utilization of locally available materials, and (c) to reduce costs through standardization of sizes. Grading specifications are carefully set by federal, state and local agencies, and industry associations. Gradation of an aggregate may be expressed as; total percent passing, total percent retained, and percent passing-retained.

Sieve sizes to be used are designated in governing specifications. Determining the percentages from weights obtained by sieve analysis is illustrated in Figure 3.5. Gradations are usually expressed on the basis of total percent passing, which indicates the total percent of aggregate by weight that will pass each given size sieve. The total percent retained is just the opposite: the total percent by weight retained on each given sieve. The percent passing-retained, two successive sieve sizes or individual percent for each size group, indicates the percent retained by weight on each sieve in the sieve analysis.

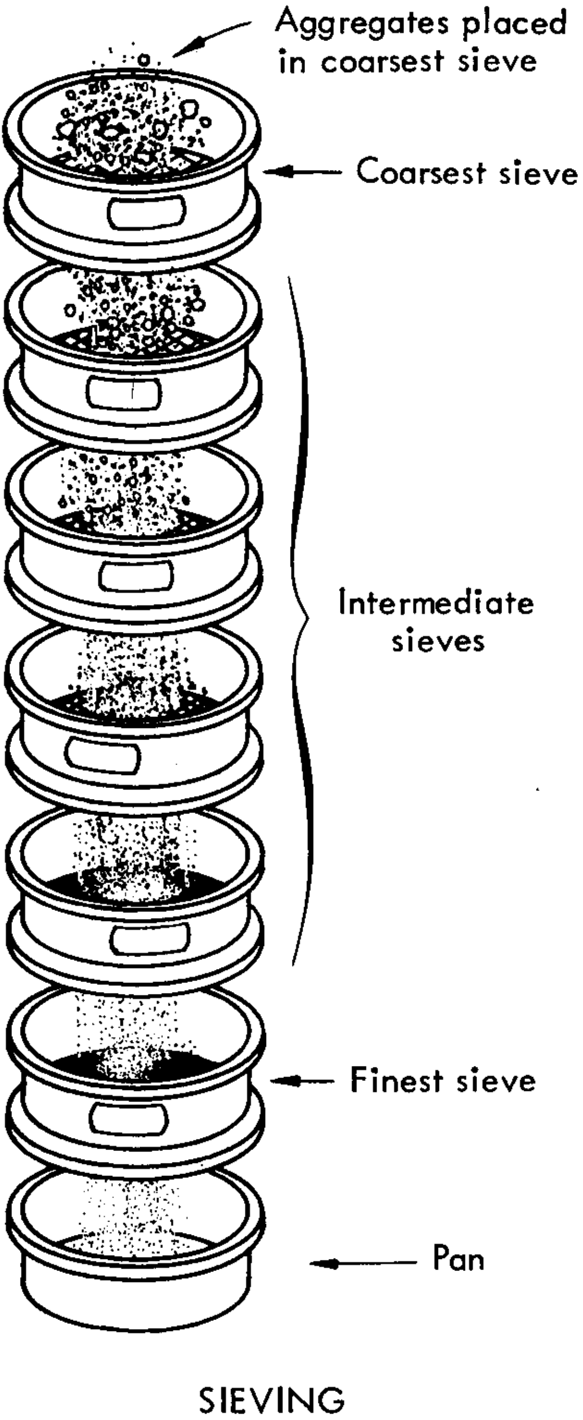


Figure 3.4 Sieve Analysis

Certain descriptive terms used in referring to aggregate gradations are as follows:

- *coarse aggregate,* all the material retained on the 2.36 mm (No. 8) sieve
- *fine aggregate,* all the material passing the 2.36 mm (No. 8) sieve
- *mineral filler, fine aggregate,* at least 70 percent of which passes the 75 μm (No. 200) sieve.

Aggregate Size Designation

All hot-mix asphalt pavement specifications require aggregate particles to be within a certain range of sizes and for each size of particle to be present in a certain proportion. This distribution of various particle sizes within the aggregate used is called the aggregate gradation or mix gradation. To determine whether or not an aggregate gradation meets specifications requires an understanding of how particle size and gradation are measured.

Sieve Size	Ret. ea. Sieve (grams)	Passing ea. sieve (grams)	Total Percent Passing	Total Percent Retained	Passing-Retained,* Percent
19.0 mm (¾-in.)	0	1135	100	0	5
12.5 mm (½-in.)	56	1079	95	5	15
9.5 mm (⅜-in.)	171	908	80	20	23
4.75 mm (No. 4)	262	646	57	43	18
2.36 mm (No. 8)	203	443	39	61	16
0.60 mm (No. 30)	182	261	23	77	6
0.30 mm (No. 50)	68	193	17	83	5
0.15 mm (No. 100)	57	136	12	88	4.5
0.075 mm (No. 200)	51	85	7.5	92.5	7.5
Pan	85				

Total = 1135

*Passing designated sieve, retained on next smaller size.

Figure 3.5 Sieve Analysis Data Converted to Aggregate Gradation.

Because specifications list a maximum particle size for each aggregate used, the size of the largest particles in the sample must be determined. There are two designations for maximum particle size:

- Maximum size, designated as the smallest sieve through which 100 percent of the aggregate sample particles pass, and
- Nominal maximum size, designated as the largest sieve that retains any of the aggregate particles but generally not more than 10 percent for the larger sized aggregates.

To illustrate the difference between the two designations, consider this example:

> A sample of aggregate to be used in a paving mixture is put through a sieve analysis. All of the aggregate particles (100 percent) pass through the 25mm (1 in.) sieve [the sieve which has openings measuring 25 mm (1 in.).] The coarsest aggregate particles are trapped on the 19 mm (¾ in.) sieve, the sieve directly below the 25 mm (1 in.) sieve. In this case, the maximum

size is 25 mm (1 in.). The nominal maximum size is 19 mm (¾ in.).

A paving mixture is classified according to either its maximum size or its nominal maximum size. Therefore, according to the maximum size of the aggregate described in the example, the mix would be termed a 25 mm (1 in.) mix; according to its nominal maximum size, the mixture would be a 19 mm (¾ in.) mix.

Sieve numbers and sizes most often used in grading aggregate for asphalt paving mixes are shown in Figure 3.6.

Sieve Size	*Mix Designation and Nominal Maximum Size of Aggregate*				
	37.5 mm (1½ in.)	*25.0 mm (1 in.)*	*19.0 mm (¾ in.)*	*12.5 mm (½ in.)*	*9.5 mm (⅜ in.)*
	Total Percent Passing (by weight)				
50 mm (2 in.)	**100**	**—**	**—**	**—**	**—**
37.5 mm (1½ in.)	**90 to 100**	**100**	**—**	**—**	**—**
25.0 mm (1 in.)	**—**	**90 to 100**	**100**	**—**	**—**
19.0 mm (¾ in.)	**56 to 80**	**—**	**90 to 100**	**100**	**—**
12.5 mm (½ in.)	**—**	**56 to 80**	**—**	**90 to 100**	**100**
9.5 mm (⅜ in.)	**—**	**—**	**56 to 80**	**—**	**90 to 100**
4.75 mm (No. 4)	**23 to 53**	**29 to 59**	**35 to 65**	**44 to 74**	**55 to 85**
2.36 mm (No. 8)*	**15 to 41**	**19 to 45**	**23 to 49**	**28 to 58**	**32 to 67**
1.18 mm (No. 16)	**—**	**—**	**—**	**—**	**—**
0.60 mm (No. 30)	**—**	**—**	**—**	**—**	**—**
0.30 mm (No. 50)	**4 to 16**	**5 to 17**	**5 to 19**	**5 to 21**	**7 to 23**
0.15 mm (No. 100)	**—**	**—**	**—**	**—**	**—**
0.075 mm (No. 200)**	**0 to 5**	**1 to 7**	**2 to 8**	**2 to 10**	**2 to 10**
Asphalt Cement, weight percent of Total Mixture***	**3 to 8**	**3 to 9**	**4 to 10**	**4 to 11**	**5 to 12**
	Suggested Coarse Aggregate Size Numbers (see Figure 3.7)				
	4 and 67 or 4 and 68	**5 and 7 or 57**	**67 or 68 or 6 and 8**	**7 or 78**	**8**

*In considering the total grading characteristics of an asphalt paving mixture the amount passing the 2.36 mm (No. 8) sieve is a significant and convenient field control point between fine and coarse aggregate. Gradings approaching the maximum amount permitted to pass the 2.36-mm (No. 8) sieve will result in pavement surfaces having comparatively fine texture, while gradings approaching the minimum amount passing the 2.36-mm (No. 8) sieve will result in surfaces with comparatively coarse texture.

**The material passing the 0.075 mm (No. 200) sieve may consist of fine particles of the aggregates or mineral filler, or both. It shall be free from organic matter and clay particles and have a plasticity index not greater than 4 when trested in accordance with ASTM Method D 423 and Method D 424.

***The quantity of asphalt cement is given in terms of weight percent of the total mixture. The wide difference in the specific gravity of various aggregates, as well as a considerable difference in absorption, results in a comparatively wide range in the limiting amount of asphalt cement specified. The amount of asphalt required for a given mixture should be determined by appropriate laboratory testing or on the basis of past experience with similar mixtures, or by a combination of both.

Figure 3.6 Typical Composition of Asphalt Concrete.

Size Number	Nominal Size, Square Openings	Amounts Finer than Each Laboratory Sieve (Square Openings), weight percent														
		100-mm (4-in.)	90-mm (3½-in.)	75-mm (3-in.)	63-mm (2½-in.)	50-mm (2-in.)	37.5 mm (1½-in.)	25.0-mm (1-in.)	19.0-mm (¾-in.)	12.5-mm (½-in.)	9.5-mm (⅜-in.)	4.75-mm (No. 4)	2.36-mm (No. 8)	1.18-mm (No. 16)	300-μm (No. 50)	150-μm (No. 100)
1	90 to 37.5-mm (3½ to 1½-in.)	100	90 to 100	...	25 to 60	...	0 to 15	...	0 to 5	...	...	...	...	...	...	...
2	63 to 37.5-mm (2½ to 1½-in.)	...	...	100	90 to 100	35 to 70	0 to 15	...	0 to 5	...	...	...	...	...	...	...
24	63 to 19.0-mm (2½ to ¾-in.)	...	...	100	90 to 100	...	25 to 60	...	0 to 10	0 to 5	...	...	...	...	...	...
3	50 to 25.0-mm (2 to 1-in.)	...	...	...	100	90 to 100	35 to 70	0 to 15	...	0 to 5	...	...	...	...	...	...
357	50 to 4.75-mm (2-in. to No. 4)	...	...	...	100	95 to 100	...	35 to 70	...	10 to 30	...	0 to 5	...	...	...	...
4	37.5 to 19.0-mm (1½ to ¾-in.)	...	...	...	...	100	90 to 100	...	20 to 55	0 to 15	...	0 to 5	...	...	...	...
467	37.5 to 4.75-mm (1½ to No. 4)	...	...	...	...	100	95 to 100	...	35 to 70	...	10 to 30	0 to 5	...	...	...	...
5	25.0 to 12.5-mm (1 to ½-in.)	...	...	...	...	...	100	90 to 100	20 to 55	0 to 10	0 to 5	...	...	...	...	...
56	25.0 to 9.5-mm (1 to ⅜-in.)	...	...	...	...	...	100	90 to 100	40 to 75	15 to 35	0 to 15	0 to 5	...	...	...	...
57	25.0 to 4.75-mm (1-in. to No. 4)	...	...	...	...	...	100	95 to 100	...	25 to 60	...	0 to 10	0 to 5	...	...	...
6	19.0 to 9.5-mm (¾ to ⅜-in.)	...	...	...	...	...	...	100	90 to 100	20 to 55	0 to 15	0 to 5	...	...	...	...
67	19.0 to 4.75-mm (¾-in. to No. 4)	...	...	...	...	...	...	100	90 to 100	...	20 to 55	0 to 10	0 to 5	...	...	...
68	19.0 to 2.36-mm (¾-in. to No. 8)	...	...	...	...	...	...	100	90 to 100	...	30 to 65	5 to 25	0 to 10	0 to 5	...	...
7	12.5 to 4.75-mm (½-in. to No. 4)	...	...	...	...	...	...	...	100	90 to 100	40 to 70	0 to 15	0 to 5	...	...	...
78	12.5 to 2.36-mm (½-in. to No. 8)	...	...	...	...	...	...	...	100	90 to 100	40 to 75	5 to 25	0 to 10	0 to 5	...	...
8	9.5 to 2.36-mm (⅜-in. to No. 8)	...	...	...	...	...	...	...	...	100	85 to 100	10 to 30	0 to 10	0 to 5	...	...
89	9.5 to 1.18-mm (⅜-in. to No. 16)	...	...	...	...	...	...	...	...	100	90 to 100	20 to 55	5 to 30	0 to 10	0 to 5	...
9	4.75 to 1.18-mm (No. 4 to No. 16)	...	...	...	...	...	...	...	...	...	100	85 to 100	10 to 40	0 to 10	0 to 5	...
10	4.75-mm (No. 4 to 0[a])	...	...	...	...	...	...	...	...	...	100	85 to 100	...	...	...	10 to 30

[a] Screenings.

Figure 3.7 Standard Sizes of Coarse Aggregate.

Sampling

The results of a sieve analysis should, of course, reflect the condition and characteristics of all the aggregate from which the sample was obtained. Unless the sample is truly representative, the examination or tests of the sample apply to the sample only and not to the entire aggregate shipment or stockpile. Accuracy in sampling, therefore, is equally as important as accuracy in testing.

Great care must be taken to obtain a truly representative sample from a stockpile of coarse and fine aggregate. Segregation usually occurs when the material is stockpiled, with coarser particles generally rolling to the base of the pile. Separate samples must be taken at various levels and locations in the pile. If a stockpile of sand, however, is to be sampled, it is usually only necessary to remove the dry layer where segregation occurs and sample the damp material below.

When samples of coarse aggregates are taken from stockpiles they should be taken at or near the top, base, and some intermediate point. A board should be shoved into the pile just above the sampling area to prevent further segregation while sampling. Fine aggregate stockpiles may be sampled with a sampling tube 30 mm (1¼ in.) in diameter and 2 m (6 ft) long. Procedures for sampling are described in ASTM D 75 and AASHTO T 2. When possible, aggregate samples should be taken from a conveyor feed belt as such samples are usually more representative than stockpile samples.

The weight of sample required for gradation analysis is based on the maximum particle size of the aggregate; the greater the maximum size, the larger the required sample weight. The following are suggested minimum sample weights for different aggregate sizes.

Max. Nominal Aggregate Size	Sample kg (lb)
2.36mm (No. 8)	10 (25)
4.75mm (No. 4)	10 (25)
9.5 mm (⅜ in.)	10 (35)
12.5 mm (½ in.)	15 (35)
19.0 mm (¾ in.)	25 (55)
25.0 mm (1 in.)	50 (110)
37.5 mm (1½ in.)	75 (165)
50.0 mm (2 in.)	100 (220)

Dry-Sieve Analysis

The dry-sieve analysis and washed-sieve analysis are two methods of determining relative proportions of various particle sizes in an aggregate sample. Standard procedures for making the dry sieve analysis are given in AASHTO T 27 and ASTM C 136. The procedure for making the dry sieve analysis is briefly outlined as follows:

1. Samples for sieve analysis are prepared by quartering methods or by using a sample splitter. Fine aggregate samples reduced by quartering are first mixed to a moist condition.

2. Aggregates containing both fine and coarse materials are separated into two sizes with the 4.75 mm (No. 4) sieve.
3. Samples are dried to a constant weight.
4. Samples are sieved separately, using a specified procedure.
5. Weights of the fractions retained on each sieve and pan are determined, as is the gradation for each sample. In cases where fine and coarse fractions are processed and they represent one sample, the two gradations are combined based on the percent passing the 4.75 mm (No. 4) sieve.

Washed-Sieve Analysis

Dry sieving alone is usually satisfactory for routine testing of normally graded aggregates. When the aggregate contains extremely fine dust or clay, which may cling to the coarser aggregate particles, a washed sieve analysis should be made. This procedure is detailed in AASHTO T 11 and ASTM C 117. The samples are prepared in the same way as for dry sieve analysis, but before sieving the following is done:

1. Sample weights are determined after drying to a constant weight.
2. Sample is placed in a container and is immersed in water containing a wetting agent.
3. Contents of the container are agitated vigorously and wash water is poured over the nested sieves.
4. Operation is repeated until wash water is clear.
5. Material retained on the sieves is returned to the washed sample and the washed aggregate dried to a constant weight.
6. Material is weighed with the loss in weight representing the amount of material finer than 75 μm (No. 200) sieve.
7. Washed sample is sieved by using dry-sieving procedure.
8. Weights are converted to percents, keeping in mind that the original dry weight before washing is the true weight for using as 100 percent.

Specific Gravity

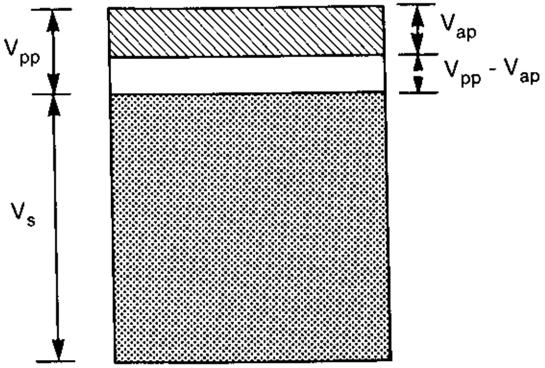
Definitions

The specific gravity of an aggregate is the ratio of the weight of a unit volume of material to the weight of the same volume of water at 20 to 25°C (68 to 77°F). There are three generally accepted types of specific gravities for aggregate, depending on the defined volume of the particle. They are:

1. Apparent specific gravity,
2. Bulk specific gravity, and
3. Effective specific gravity.

Referring to Figure 3.8 apparent specific gravity considers the volume as being the volume of the aggregate itself. It does not include the volume of any pores or capillaries that become filled with water after a 24-hour soaking. Bulk specific gravity considers the overall volume of the aggregate particle, including the pores that become filled with water after a 24-hour soaking. The effective specific gravity considers the overall volume of the aggregate exclusive of the volume of pores that absorbs asphalt and can be approximated as the average of the apparent and bulk specific gravities.

Air voids in the compacted asphalt pavement course appear throughout the mix as small pockets of air between the asphalt coated aggregate particles. The choice of specific gravity for an aggregate used in asphalt mix calculations could have substantial effect upon the calculated amount of air voids in the compacted pavement. The actual specific gravity of the aggregate in the mixture depends upon the degree to which the aggregate absorbs asphalt. When apparent specific gravity is used, asphalt is assumed to be absorbed by all the water-permeable pores. If bulk specific gravity is used, it is assumed the asphalt is not absorbed by the water permeable pores.



Oven-dry weight of aggregate = W_s
Unit weight of water, $\gamma_w = 1 g/cm^3$

(For simplicity, metric measurements are typically used in these calculations.)

V_s = Volume of solids
V_{pp} = Volume of water permeable pores
V_{ap} = Volume of pores absorbing asphalt
$V_{pp} - V_{ap}$ = Volume of water permeable pores not absorbing asphalt

$$\text{Apparent specific gravity} = G_{sa} = \frac{W_s}{V_s \quad \gamma_w}$$

$$\text{Bulk specific gravity} = G_{sb} = \frac{W_s}{(V_s + V_{pp})\gamma_w}$$

$$\text{Effective specific gravity} = G_{se} = \frac{W_s}{(V_s + V_{pp} - V_{ap})\gamma_w}$$

Figure 3.8 Relationship Between the Different Specific Gravities of an Aggregate Particle.

Except in rare cases, neither is true. The concept of effective specific gravity more nearly approaches the correct value for determining the voids in a compacted asphalt paving mix. Bulk specific gravity may be used, however, if an allowance for the asphalt absorbed by the aggregate is taken in to account.

For combining aggregates, quantities of the aggregate components may require adjusting because of differing specific gravities. When this is necessary, the bulk specific gravities are usually used in the computations.

Determinations for Coarse Aggregate

Equipment and procedures for determining the specific gravities of coarse aggregates are outlined in AASHTO T 85 and ASTM C 127. The method is briefly outlined as follows:

1. About 5 kg of thoroughly washed aggregate retained on a 4.75 mm (No. 4) sieve is oven-dried to a constant weight.
2. The dried sample is immersed in water for 24 hours.
3. The aggregate is removed from the water and surface-dried until all visible films of water are removed, although the surfaces still appear damp.
4. The weight of the sample in the surface-dry condition is obtained.
5. The saturated surface-dry sample is placed in a wire basket and its submerged weight in water is determined.
6. The sample is then oven-dried to a constant weight and the weight recorded.
7. The specific gravities are calculated as follows, where:

A = oven-dried weight of aggregate, g.
B = saturated surface-dry weight of aggregate, g, and
C = submerged weight of aggregate in water, g.

then

$$\text{Apparent specific gravity, } G_{sa} = \frac{A}{A-C} \tag{1}$$

$$\text{Bulk specific gravity, } G_{sb} = \frac{A}{B-C} \tag{2}$$

$$\text{Absorption} = \frac{(B-A)\,100}{A} \tag{3}$$

Determinations for Fine Aggregate

Equipment and procedures for determining the apparent and bulk specific gravities of fine aggregates are detailed in AASHTO T 84 and ASTM C 128. The procedure is briefly discussed as follows:

1. Approximately 1000 g of fine aggregate is oven-dried to a constant weight.
2. The material is immersed in water for 24 hours.
3. Sample is then spread on a flat surface and exposed to a gently moving current of warm air.
4. A saturated surface-dry condition is reached when the material slumps when an inverted cone is removed in which a sample of material was lightly compacted.
5. A 500 g sample of the material in the saturated surface-dry condition is placed in a flask. The flask is then filled with water, using a specified procedure and weighed.
6. Fine aggregate is removed from the flask, oven-dried to a constant weight, and then weighed.
7. The specific gravities are calculated as follows, where:

A = weight of oven-dry sample, g
B = weight of pycnometer filled with water, g
C = weight of pycnometer with specimen and water to calibration mark, g

$$\text{Apparent specific gravity} = G_{sa} = \frac{A}{B+A-C} \tag{4}$$

$$\text{Bulk specific gravity} = G_{sb} = \frac{A}{B+500-C} \tag{5}$$

$$\text{Absorption} = \left(\frac{500-A}{A}\right)100 \tag{6}$$

Effective Specific Gravity

The procedure for determining the effective specific gravity does not have an AASHTO or ASTM designation. Data obtained for determining the maximum specific gravity of asphalt mixtures (ASTM D 2041) may be used, however, in making the calculation.

The procedure involves mixing a known amount of aggregate and asphalt together, using accepted mixing procedures. After the mixture has cooled, the absolute volume of the material is determined by weighing the mixture first in air, then submerged, and recording the difference in weights.

The formula for calculating the effective specific gravity of aggregate is as follows:

$$G_{se} = \frac{P_s}{\dfrac{P_{Mix} - P_b}{G_{mm} \quad G_b}}$$

where

P_s = percent aggregate by total weight of mix
P_b = percent asphalt by total weight of mix
P_{mix} = weight of mix, 100%
$P_b + P_s = 100\%$
G_{mm} = theoretical maximum specific gravity as determined by ASTM D 2041
G_b = specific gravity of asphalt cement

The calculations are treated in more detail when asphalt mix design calculations are discussed.

Surface Area

Function

When asphalt paving mixes are designed using the Hveem method of mix design, a starting point for asphalt content is found by using the surface area method. The surface area theory may be expressed as:

$$\text{Percent asphalt} = 100 \, A t \, \gamma_b$$

where

A = surface area of the aggregate in m^2/kg (ft^2/lb),
t = thickness of the asphalt film coating, m (ft), and
γ_b = unit weight of asphalt, kg/m^3 (lb/ft^3).

Calculations

The Hveem method for the laboratory design of asphalt paving mixtures (described in Chapter 4) uses the surface area calculation and absorption test to select an asphalt content as a starting point in the test series. The surface area of the aggregate is determined by multiplying the percentages of material passing a given set of sieves by a set of surface area factors and adding the products. Figure 3.9 shows how the surface area is determined for a given aggregate when its gradation is known. It is important to note that all the sieve sizes shown must be used in this method.

Sieve Size	Percent Passing	×	Surface Area Factor m^2/kg (ft^2/lb)	=	Surface Area m^2/kg (ft^2/lb)
19.0 mm (¾ in.)	100		—		
12.5 mm (½ in.)	95		— *		.41 (2.0)
9.5 mm (⅜ in.)	80		—		
4.75 mm (No. 4)	57		.41 (2)		.23 (1.1)
2.36 mm (No. 8)	39		.82 (4)		.32 (1.6)
1.18 mm (No. 16)	31		1.64 (8)		.51 (2.5)
600 μm (No. 30)	23		2.87 (14)		.66 (3.2)
300 μm (No. 50)	17		6.14 (30)		1.04 (5.1)
150 μm (No. 100)	12		12.29 (60)		1.47 (7.2)
75 μm (No. 200)	7.5		32.77 (160)		2.46 (12.0)
					Σ7.1 (34.7)

*Surface area factor of plus 4.75 mm (No. 4) material is .41 (2)

Figure 3.9 Calculation of Surface Area.

3.3 Aggregate Gradation Blending Calculations

Introduction

Reasons for Proportioning Gradations

Combining two or more aggregates having different gradations to produce an aggregate blend that meets grading specifications for a particular asphalt mix is common in asphalt construction. Determining relative amounts of various aggregates needed to obtain a desired gradation is a problem in proportioning that can be solved by a number of methods.

Basic Formula

Regardless of the number of aggregates combined or of the method by which the proportions are determined, the basic formula expressing the combination is:

$$P = Aa + Bb + Cc \text{ etc.} \tag{1}$$

where

P = the percentage of material passing a given sieve for the combined aggregates A, B, C, etc.;

A, B, C, etc. = percentages of material passing a given sieve for aggregates A, B, C, etc.; and

a, b, c, etc. = proportions of aggregates, A, B, C, etc. used in the combination and where the total = 1.00.

The combined percentages, P, in Eq. 1 should closely agree with the desired percentages for the combination for the different sieve sizes. None should fall outside the established grading specification limits. Obviously, there may be several acceptable combinations. An optimum combination would be one in which the percentages of the blend are in as close agreement as possible to the desired percentages originally set up.

Sophisticated mathematical procedures have been developed that will determine an optimum combination of aggregates. Even with a calculator, however, these methods are involved and time-consuming. Usually a trial-and-error method guided by a certain amount of reasoning is the easiest procedure to determine a satisfactory combination. The computer program *Computer-Assisted Asphalt Mixture Analysis* (CAMA), Asphalt Institute, is capable of blending up to ten different aggregates. CAMA can optimize the blend with respect to the upper & lower control limits, as well as develop a least cost blend that will meet the specification requirements.

Proportioning Determinations

Combining Two Aggregates

The basic formula (Eq. 1) for combining two aggregates is

$$P = Aa + Bb \tag{2}$$

Since a+b=1, then a=1-b. Substituting this into Eq. 2, and solving for b:

$$b = \frac{P-A}{B-A} \tag{3}$$

An expression for, a, can also be found:

$$a = \frac{P-B}{A-B} \tag{4}$$

Assume that a single aggregate stockpile is to be blended with sand to meet grading requirements for an asphalt paving mixture. These are given in Figure 3.10a as aggregates A and B. To make a determination:

1. Examine the two gradations to determine which aggregate will contribute most of the material for certain sizes. In this case, most of the minus 2.36 mm (No. 8) aggregate will be furnished by aggregate B.
2. Using the percentages for the 2.36 mm (No. 8) sieve and substituting into Eq. 3, the proportions are determined to meet the midpoint of the specification (Fig. 3.10b).
3. Inspection of the blended gradation shows the percent passing 75 μm (No. 200) close to the lower specification limit. Increasing the proportion of aggregate B to .55 and recomputing, yields the blended gradation shown in Figure 3.10c.
4. Inspection now shows the gradation to border the upper limit of the 600 μm (No. 30) specification. A final adjustment should be made by reducing the proportion of B to .52 or .53.

Combining Three Aggregates

Assume that mineral filler, C, is to be blended with aggregates A and B to obtain a gradation meeting specification requirements. The specification and gradations are given in Figure 3.11a.

1. An inspection of the gradations indicates that there is a reasonably clean separation between the plus 2.36 mm (No. 8) sizes and minus 2.36 mm (No. 8) sizes. Aggregate A will furnish most of the plus 2.36 mm (No. 8) sizes.
2. Determine the approximate proportion of aggregate A required to obtain 4.25 percent passing the 2.36 mm (No. 8) sieve, using Eq. 4, (Fig. 3.11b).
3. The percentages passing 75 μm (No. 200) sieve are examined next. Values are substitutes into Eq. 1. The remainder of the calculations are self-explanatory.

Percent Passing

	19.0 mm	12.5 mm	9.5 mm	4.75 mm	2.36 mm	600 μm	300 μm	150 μm	75 μm
Sieve	¾"	½"	⅜"	No. 4	8	30	50	100	200
Spec.	100	80-100	70-90	50-70	35-50	18-29	13-23	8-16	4-10
Aggr. A	100	90	59	16	3.2	1.1	0	0	0
Aggr. B	100	100	100	96	82	51	36	21	9.2

$$(\text{No. 8}),\ b = \frac{P - A}{B - A} = \frac{42.5 - 3.2}{82 - 3.2} = 0.50,\ a = 1\text{-}050 = 0.50$$

(a) Grading Specification and sieve analysis of aggregates

Percent Passing

	19.0 mm	12.5 mm	9.5 mm	4.75 mm	2.36 mm	600 μm	300 μm	150 μm	75 μm
Sieve	¾"	½"	⅜"	No. 4	8	30	50	100	200
.50 × A	50	45.5	29.5	8.0	1.6	0.6			
.50 × B	50	50.0	50.0	48.0	41.0	25.0	**18.0**	**10.5**	**4.6**
Total	100	95.5	79.5	56.0	42.6	25.6	**18.0**	**10.5**	**4.6**
Spec.	100	80-100	70-90	50-70	35-50	18-29	**13-23**	**8-16**	**4-10**

Minus No. 200 low, increase b to 0.55, a to 0.45
(b) First trial combination

Percent Passing

	19.0 mm	12.5 mm	9.5 mm	4.75 mm	2.36 mm	600 μm	300 μm	150 μm	75 μm
Sieve	¾"	½"	⅜"	No. 4	8	30	50	100	200
.45 × A	45	40.5	26.6	7.2	1.4	0.5			
.55 × B	55	55.0	55.0	52.8	45.1	28.0	19.8	11.5	5.1
Total	100	95.5	81.6	60.0	46.5	28.5	19.8	11.5	5.1
Spec.	100	80-100	70-90	50-70	35-50	18-29	13-23	8-16	4-10

Minus No. 30 high, let b = 0.52, a = 0.48
(c) Second trial combination

Figure 3.10 Trial-and-Error Calculations for Combining Two Aggregates.

Should the blended gradation fall outside the specification limits, that proportion in the blend apparently responsible should be altered, with the other proportions altered to make up a total of 100 percent. Sometimes it is necessary to delete a proposed aggregate and obtain replacement material if a satisfactory blend cannot be developed with the aggregate initially suggested.

Trial-and-error solutions are just that. An inspection of the gradations for indications to assist in establishing proportions narrows the number of solutions considerably.

Percent Passing

	19.0 mm	12.5 mm	9.5 mm	4.75 mm	2.36 mm	600 μm	300 μm	150 μm	75 μm
Sieve	**¾"**	**½"**	**⅜"**	**No. 4**	**8**	**30**	**50**	**100**	**200**
Spec.	100	80-100	70-90	50-70	35-50	18-29	13-23	8-16	4-10
Aggr. A	100	90	59	16	3.2	1.1			
Aggr. B	100	100	100	96	82	51	36	21	9.2
Aggr C.	100	100	100	100	100	100	98	93	82

(a) Grading specification and gradations

$$(\text{No. 8}),\ b = \frac{P - B}{A - B} = \frac{4.25 - 82}{3.2 - 82} = 0.50$$

$$\begin{aligned} (\text{No. 200}),\quad & P = Aa \times Bb = Cc \\ & 7 = (0.50) + 9.2b + 82c,\ b + c = 1 - 0.50 = 0.50 \\ & 7 = 9.2\,(0.50 - c) \times 82c \end{aligned}$$

$$c = \frac{7 - 4.6}{82 - 9.2} = b = 0.50 - 0.03 = 0.47$$

Percent Passing

	19.0 mm	12.5 mm	9.5 mm	4.75 mm	2.36 mm	600 μm	300 μm	150 μm	75 μm
Sieve	**¾"**	**½"**	**⅜"**	**No. 4**	**8**	**30**	**50**	**100**	**200**
.50 × A	50	4.50	29.5	8.0	1.6	0.6			
.47 × B	47	47.0	47.0	45.1	38.5	24.0	16.9	9.9	4.3
.03 × C	3	3.0	3.0	3.0	3.0	3.0	3.0	2.8	2.5
Total	100	95.0	79.5	56.1	43.1	27.6	19.9	12.7	6-8
Spec.	100	80-100	70-90	50-70	35-50	18-29	13-23	8-16	4-10

(b) First trial solution for combining three aggregates

Figure 3.11 Trial-and-Error Solution for Combining Three Aggregates.

Grading Adjustments

Adjusting for Different Specific Gravities

Aggregate gradations and grading curves are determined and expressed in precentages of total weight. Grading specifications, however, are established to meet volumetric requirements in the asphalt paving mix. As long as the specific gravities of the combined aggregate materials are reasonably alike, the percentages by weight may be interpreted as percentages by volume for all practical purposes. When the specific gravities of the individual aggregates differ significantly (usually by 0.20 or more), the aggregate proportions should be adjusted. The adjustment is based on the fact that:

Volume × Specific Gravity × Unit Weight of Water = Weight

$$V \times G_s \times \gamma_w = W$$

Assume the following combination was calculated for three aggregates having different specific gravities:

Aggregate	Sp Gr	*Proportion*	$\gamma_w = 1 g/cm^3$
A	1.00	0.52	
B	2.00	0.45	
C	3.00	0.03	

Calculations for adjustments are shown in Figure 3.12.

Aggr.	% Vol.	Sp Gr	Weight	% Wt.
A	52.0	1.00	52.00	34.4
B	45.0	2.00	90.00	59.6
C	3.0	3.00	9.00	6.0
Total	100.0	—	151.00	100.0

(1) Weight = Vol. × Sp Gr × γw

(2) $\% \text{ Wt.} = \dfrac{\text{Individual Weight}}{\text{Total Weight}} + 100 = \dfrac{W}{151} + 100$

Figure 3.12 Adjusting Percentages by Volume to Percentages by Weight.

Adjusting by Wasting

Where the main source of aggregate is local roadside pit, it is often the case that crusher-run aggregates are either coarser or finer than desired. Finer material can usually be blended to achieve a gradation that is coarser than desired. But for gradations that have an excess of fines, the most economical adjustment is usually made by wasting a portion of the fine fraction. Most crushing plants will make the separations on the 4.75 mm (No. 4) or possibly

2.36 mm (No. 8) screen. Where an excess on a smaller size occurs, the correction is made by wasting a portion of the minus 4.75 mm (No. 4) fraction. The amount of waste is expressed as a percent, considering the total crusher-run material as 100 percent.

The formulas for analysis of gradations before and after wasting are as follows:

(Sizes above waste screen)

$$R_b = \frac{R_2 R_a}{R_1} \tag{5}$$

(Sizes below waste screen)

$$P_b = \frac{P_2 P_a}{P_1} \tag{6}$$

where

P_a, R_a = percent passing, or retained, of given size before wasting;
P_b, R_b = adjusted percent passing, or retained, of given size after wasting;
P_1, R_1 = percent passing, or retained, of waste size before wasting; and
P_2, R_2 = percent passing, or retained, of waste size after washing.

The percent of waste, W, is found as follows:

$$W = \frac{(P_1 - P_2)\,100}{(100 - P_2)} \tag{7}$$

Assume that a single aggregate stockpile is being produced from a local roadside pit. The specification limits and crusher-run gradation are shown in Figure 3.13. Note that the 4.75 mm (No. 4) size is above specification limits and that the other percentages approach the upper limits of the specification. A portion of the minus 4.75 mm (No. 4) fraction will be wasted to reduce the percent passing 4.75 mm (No. 4) from 75 to 70. The percentages of the minus 4.75 mm (No. 4) sizes are found using Eq. 6 as shown. The percentages of the plus 4.75 mm (No. 4) sizes are first converted to percent-retained and the adjusted percent retained on 4.75 mm (No. 4) determined. The percentages of coarse sizes are found using Eq. 5 and converting to percent passing.

Another way to look at the problem is as follows. The original aggregate sample can be divided into material that is retained on the 4.75 mm (No. 4) sieve and material that passes the 4.75 mm (No. 4) sieve. If the original sample weighed 50 kg (110 lb), then 12.5 kg (27.5 lb) or 25% of the material is retained on the 4.75 mm (No. 4) sieve and 37.5 kg (82.5 lb), or 75% of the material passes the 4.75 mm (No. 4) sieve. To bring the gradation into specification, a portion of the minus 4.75 mm (No. 4) material must be wasted. To calculate the percent of wasted material, recall that in the new or adjusted gradation, 12.5 kg (27.5 lb) now represents 30% of the sample, as shown in Figure 3.13. Now, one must determine the weight of the minus 4.75 mm (No. 4) material, which represents 70% of the sample. This can be calculated by the equation shown below, where X represents the weight of the minus 4.75 mm (No. 4) material in the new sample.

$$\frac{12.5\text{ kg } (27.5\text{ lb})}{30\%} = \frac{X\text{ lb}}{70\%} \tag{8}$$

$$X = 29.2\text{ kg } (64.2\text{ lb})$$

Note that the original sample contained 37.5 kg (82.5 lb) of the minus 4.75 mm (No. 4) material and after wasting the sample contains 29.2 kg (64.2 lb) of minus 4.75 mm (No. 4) material. The difference between the original and final weights 8.3 kg (18.3 lb) represents the minus 4.75 mm (No. 4) material that was wasted. Therefore, the percent wasted can be calculated as follows:

$$\frac{\text{Weight of Material Wasted}}{\text{Original Weight of Sample}} = \frac{8.3\text{ kg } (18.3\text{ lb})}{50\text{ kg } (110\text{ lb})} (100) = 16.7\% \tag{9}$$

	19.0 mm	12.5 mm	9.5 mm	4.75 mm	2.36 mm	600 μm	150 μm	75 μm
Sieve	¾"	½"	⅜"	No. 4	8	30	100	200
Spec.	100	80-100	70-90	55-73	40-55	20-30	10-18	4-10
% Pass, Pa	100	98	87	75	54	28	17	9
% Ret, Ra	0	2	13	25				
Adj % Ret, Rb	0	2	16	30				
Adj % Pass, Pb	100	98	84	70	50	26	16	8.4

$$P_b = \frac{P_2}{P_1} P_a = \frac{70}{75} P_a = 0.93\, P_a \qquad R_b = \frac{R_2}{R_1} R_a = \frac{30}{25} R_a = 1.20\, R_a$$

$$\text{Waste, } W = \frac{100\,(P_1 - P_2)}{(100 - P_2)} = \frac{100\,(75-70)}{(100-70)} = 16.7\ \%$$

[Calculations based on adjusting percent passing 4.75 mm (No. 4)]

Figure 3.13 Adjusting Gradation by Wasting.

Chapter 4
Hot-Mix Asphalt Mix Design

This chapter describes the combining of two materials (asphalt cement and aggregate) in proportions that produce satisfactory hot paving mixes. Various mix design methods and practices are discussed. Included are several calculations for evaluating mix properties in terms of a density-voids analysis.

SECTION 4.1 Properties of Asphalt Hot Mixes
☐ Introduction
☐ Considerations in Mix Design
☐ Mix Design Procedures

SECTION 4.2 Procedures for Analyzing a Compacted Paving Mixture
☐ Introduction
☐ Procedure Outline
☐ Paving Mixture Data for Sample Calculations
☐ Bulk Specific Gravity of Aggregate
☐ Effective Specific Gravity of Aggregate
☐ Maximum Specific Gravities of Mixtures with Different Asphalt Contents
☐ Asphalt Absorption
☐ Effective Asphalt Content of a Paving Mixture
☐ Percent VMA in Compacted Paving Mixture
☐ Calculation of Air Voids in Compacted Mixture

SECTION 4.3 Asphalt Hot Mix Design Methods
☐ Introduction
☐ Marshall Method of Mix Design
☐ Hveem Method of Mix Design

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COMPUTER SOFTWARE

1. *Computer-Assisted Asphalt Mixture Analysis (CAMA),* CP-6, Asphalt Institute

4.1 Properties of Asphalt Hot Mixes

Introduction

Hot-Mix Defined

Hot-mix asphalt (HMA) consists of a combination of aggregate uniformly mixed and coated with asphalt cement. To dry the aggregates and to obtain sufficient fluidity of asphalt cement for proper mixing and workability, both the aggregate and asphalt must be heated prior to mixing—hence the term hot mix.

Aggregates and asphalt are combined in a mixing facility in which all of the constituent materials are heated, proportioned, and mixed to produce the desired paving mixture. After plant mixing is complete, the hot mix is transported to the paving site and spread in a loosely compacted layer to a uniform, even surface with a paving machine. While the paving mixture is still hot, the material is further compacted by heavy motor-driven rollers to produce a smooth, well-consolidated pavement layer.

Classification of Hot Mixes

Hot-mix asphalt paving mixtures may be produced from a wide range of aggregate combinations, each having its own particular characteristics suited to specific design and construction uses. In addition to the amount and grade of asphalt used, the principal characteristics of the mix are determined by the relative amounts of

(a) coarse aggregate, retained on 2.36 mm (No. 8 sieve);
(b) fine aggregate, passing 2.36 mm (No. 8 sieve); and
(c) mineral filler, passing 75 μm (No. 200 sieve).

Asphalt concrete is a type of hot mix that meets strict requirements. It is a high-quality, carefully controlled hot mixture of asphalt cement and well-graded, high-quality aggregate thoroughly compacted into a uniform dense mass typified by dense-graded paving mixes.

Considerations in Mix Design

Desired Properties

Current mix design procedures are considered to be empirical, that is, they are based on test values that have been established on the basis of observed field performance. Because there is no quantifiable relationship between the test values and a specific level of pavement performance, it could be argued that current mix design is a combination of art and science.

Research is currently underway to develop a mechanistic design procedure which will relate mix properties to field performance. The existing mix design procedures, however, have a long history of acceptable use. When properly applied, these procedures provide a satisfactory job mix formula and quality control system.

To properly design a dense graded asphalt paving mixture for a specific application irrespective of the laboratory procedure employed, consideration must be given to the following desirable mix properties:

- Stability
- Durability
- Flexibility
- Fatigue resistance
- Skid resistance
- Impermeability
- Workability

Stability is the ability of asphalt paving mixture to resist deformation from imposed loads. Unstable pavements are marked by channeling (ruts), and corrugations (washboarding). Stability is dependent upon both internal friction and cohesion.

Internal friction is dependent on surface texture, gradation of aggregate, particle shape, density of mix, and quantity and type of asphalt. Stability is a function of the frictional and interlocking resistance of the aggregate in the mix. Frictional resistance increases with the surface roughness and angularity of the aggregate particles. It also increases with the area of particle contact. Interlocking resistance is dependent upon particle size and shape. For any given aggregate, the stability increases with the density of the compacted mass. Excessive asphalt in the mix tends to act as a lubricant and lowers the internal friction of the stone framework thereby reducing the stability of the mixture.

Cohesion is the internal binding force that is inherent in the asphalt paving mixture. The asphalt serves to maintain contact points developed between aggregate particles. Cohesion varies directly with the rate of loading, loaded area, and viscosity of the asphalt which varies inversely with the temperature. Cohesion increases with increasing asphalt content up to an optimum point and then decreases.

Durability is the property of an asphalt paving mixture that describes its ability to resist the detrimental effects of air, water, temperature and traffic. Included under weathering are changes in the characteristics of asphalt, such as oxidation and volatilization, and changes in the pavement and aggregate due to the action of water, including freezing and thawing.

Durability is generally enhanced by high asphalt contents, dense aggregate gradations, and well-compacted, impervious mixtures. One argument for an increased amount of asphalt is the resultant thicker asphalt film coating the aggregate particles. Thicker films are more resistant to age-hardening, i.e., a longer period of time is required to reduce a thicker film of asphalt to the same degree of brittleness as a thin film. Another favorable argument is that the increased amount of asphalt reduces the pore sizes of the interconnected voids, or seals them off in a mixture—making it more difficult for air and water to permeate the mix. But extreme caution must be exercised since the greater film thickness may produce a mix that is more prone to rutting and shoving.

To resist the action of water the same requirements—dense-graded aggregates, high asphalt contents, and adequate compaction—apply. It is desirable to use aggregates that retain an asphalt coating in the presence of water. A dense mixture is more resistant to displacement of asphalt from aggregate. If however, the aggregate is of a hydrophilic nature (water loving) it may be necessary to incorporate an anti-strip additive into the asphalt cement.

Sufficient asphalt must be incorporated in the mix to provide bonding properties adequate to resist the abrasive forces of traffic. Insufficient asphalt may result in aggregate being dislodged from the surface—this is known as raveling. Abrasion may also take place if the asphalt has become brittle. Overheating of asphalt in the hot-mixing process is a cause of later brittleness, leading to abrasion of the actual pavement.

While a mixture having a high asphalt content, with voids completely filled with asphalt, may provide the ultimate in durability, this would be undesirable from the standpoint of stability. When placed in the roadway, the pavement would channel and move under traffic. Bleeding or flushing of asphalt to the surface would also take place.

Maximum stability is not reached in an asphalt/aggregate mass until the amount of asphalt coating the particles has reached some critical value. Additional asphalt then tends to act as a lubricant rather than a binder, reducing the stability of the mixture and increasing the durability. It is therefore frequently necessary to compromise, keeping the asphalt content as high as possible while maintaining adequate stability.

Flexibility is the ability of an asphalt paving mixture to be able to bend slightly, without cracking, and to conform to gradual settlements and movements of the base and subgrade. Differential settlements in fill embankment occasionally occur. It is extremely difficult to develop uniform density in the subgrade during construction such that all sections have exactly the same support value. Therefore, the asphalt pavement must have the ability to conform to minor localized and differential settlements without cracking. Generally, flexibility of the asphalt paving mixture is enhanced by high asphalt contents and relatively open-graded aggregates, but the overall mix design criteria must be balanced to achieve a satisfactory job mix formula.

Fatigue Resistance is the ability of asphalt pavement to withstand repeated flexing caused by the passage of wheel loads. Tests have shown that the quantity of asphalt is extremely important when considering the fatigue resistance of a pavement mix. As a rule, the higher the asphalt content, the greater the fatigue resistance. Tests indicate that dense-graded asphalt mixes have more fatigue resistance than open-graded mixes. Well-graded aggregates that permit higher asphalt content without causing flushing or bleeding in a compacted pavement should be incorporated in the mix.

Skid Resistance is the ability of asphalt paving surface, particularly when wet, to offer resistance to slipping or skidding. The factors for obtaining high skid resistance are generally the same as those for obtaining high stability. Proper asphalt contents and aggregates with a rough surface texture are the greatest contributors to good resistance. While aggregates that have a tendency to polish may be used in underlying courses they should be avoided in surface courses. Aggregates containing non-polishing minerals with different wear or abrasion characteristics provide continuous renewal of the pavement's texture, maintaining a skid-resistance surface.

Mixes so rich in asphalt as to fill the voids in the compacted pavement will probably result in asphalt bleeding to the surface. Free asphalt on the pavement surface is undesirable and can cause slippery conditions when the pavement is wet.

Permeability is the resistance that an asphalt pavement has to the passage of air and water into or through the pavement. While the void content may be an indication of the susceptibility of a compacted paving mixture to the passage of air or water, of more significance is the interconnection of the voids and their access to the surface. Imperviousness to air and water is also necessary for durability in asphalt paving mixtures.

Workability is the ease with which paving mixtures may be placed and compacted. With careful attention to proper mix design and with the use of machine spreading, workability is not a problem. At times, the properties of aggregates that promote high stability make asphalt mixtures containing these aggregates difficult to spread or compact. Since workability problems are discovered most frequently during the paving operation, it is at this point that mix design adjustments might have to be made quickly to allow the job to proceed as efficiently as possible.

Objectives

Designing asphalt paving mixes, as with other engineering materials designs, is largely a matter of selecting and proportioning materials to obtain the desired qualities and properties in the finished construction. The overall objective is to determine an economical blend and gradation of aggregates (within the specification limits) and a corresponding asphalt content that yields a mix having the following properties:

- Sufficient asphalt to ensure a durable pavement.
- Sufficient mix stability to satisfy the traffic demands without distortion or displacement.
- Sufficient air voids in the total compacted mix to allow for thermal expansion of the asphalt and aggregate under summer temperatures

without flushing, bleeding and loss of stability, yet low enough to keep out harmful air and moisture.

- Sufficient workability to permit efficient placement of the mix without segregation.

Applications of Mix Design Testing

Normally, mix design testing has three important applications with respect to the overall construction project. These are preliminary design testing, job-mix control, and routine construction control.

Preliminary Design Testing—The principal purpose of preliminary design testing is to determine if prospective sources can provide aggregate of satisfactory quality to produce a paving mix satisfying both the gradation and mix design requirements of the specifications. Asphalt also should meet specification requirements and be compatible with the available aggregates. Initial trial mixes should be made with an aggregate gradation approaching the median of the grading limits. Where several sources of aggregate are being considered, trial blends can determine the most economical combination to use. The results of the preliminary mix design serve as a basis for making a preliminary cost estimate. These results also indicate whether the design requirements can be obtained within the framework of the specifications.

Source-acceptable testing is often performed after the contact has been awarded and after the contractor has indicated the proposed sources of all aggregate materials and asphalt. The main objective of this test is to determine the most economical blend of aggregates that will satisfy both the gradation and mix-design requirements. This assures the selection of proper materials and permits the contractor to begin stockpiling these materials at the job site.

Job-Mix Control—Job-mix control testing is performed at the start of plant production and in conjunction with calibrating the central mixing plant for the job-mix formula. The job-mix formula identifies the gradation of the combined aggregate materials and the selected asphalt content. In producing asphalt paving mixture in a plant, certain variations in the mix are inevitable. The job-mix formula has tolerances that allow reasonable variations in the gradation and asphalt content without the need for adjustments.

In the case of batch type mixing facilities, initial trial mixes for job-mix control testing may be made on aggregate samples obtained from the hot bins of the plant. Such testing serves to determine whether the paving mixture produced under the job-mix formula meets all specification requirements.

Routine Construction Control—Construction control testing is performed as a routine and periodic item of inspection during paving construction. Random samples of the hot paving mixture are obtained at the central mixing facility and check-tested for design properties. The results of these tests are compared with the job-mix control tests and the overall specification requirements. Where irregularities occur and the limits of the job-mix formula are exceeded, appropriate corrections are required at the plant. Although this type of mix design testing is routine in nature, it requires close attention. Its importance is emphasized because the results serve as a basis for the final acceptance or rejection of the paving construction.

Mix Design Procedures

General procedures for designing asphalt paving mixes are discussed in the following paragraphs. Specific mix design methods for the Marshall and Hveem procedures are described in Section 4.3.

Selecting and Combining Aggregates

For contract work, the contractor usually selects the aggregates himself. Locally available materials such as gravel and sands may be considered. The engineer may save in the cost of producing asphalt paving mixes if he determines an economical combination of aggregates meeting quality requirements that, when combined, produce the desired gradation.

The procedures for proportioning aggregates to produce a combined gradation were discussed in Chapter 3.

Specific Gravity Determinations

Procedures for determining the specific gravity of asphalt and the specific gravities of aggregates have already been discussed. Generally, the following specific gravities are used:

- Asphalt cement: apparent specific gravity @ 25°C (77°F), G_b
- Coarse aggregate: bulk specific gravity, G_{sb}
- Fine aggregate: bulk specific gravity, G_{sb}
- Mineral filler: apparent specific gravity, G_{sa}

The specific gravity of the blended aggregate is found by:

$$G_{sb} = \frac{100}{\frac{P_1}{G_1} + \frac{P_2}{G_2} \dots + \frac{P_n}{G_n}} \tag{1}$$

where

G_{sb} = Bulk specific gravity of the blend
$P_1, P_2 \dots P_n$ = Respective percents of aggregates 1, 2, etc., and $P_1 + P_2 + \dots + P_n = 100$
$G_1, G_2 \dots G_n$ = Respective bulk specific gravities of aggregates 1, 2, etc.

Preparing Trial Specimens

Compacted specimens of asphalt paving mixtures are prepared in the laboratory. A number of these specimens are made at varying asphalt contents. The exact method of preparing and compacting the specimens is spelled out in the test method employed. Two widely accepted methods of mix design testing are the Marshall Method (ASTM D 1559) and the Hveem Method (ASTM D 1560, D 1561), see Section 4.3.

Specific Gravity of the Specimen

After the laboratory specimen has been compacted, extruded from its mold, and cooled, the bulk specific gravity is determined by several weighings of the specimen prior to further testing (ASTM D 1188). Briefly, the procedure involved is as follows:

1. Weigh specimen in air;
2. Coat specimen with paraffin;
3. Weigh coated specimen in air, and
4. Weigh coated specimen submerged in water.

For compacted specimens of asphalt concrete, sheet asphalt, and other mixes that have a fine surface texture or "tight" surface, omit steps 2 and 3. Instead, the saturated surface dry weight may be obtained following the submerged weight, as detailed in ASTM D 2726.

Steps 2 and 3 are definitely necessary if the surface of the specimen is porous or will permit the infiltration of water during the submerged weighing.

Stability Tests

The stability of each specimen is determined according to the mix design method being used. This is a destructive test, so all weighings and measurings required must be made prior to this test.

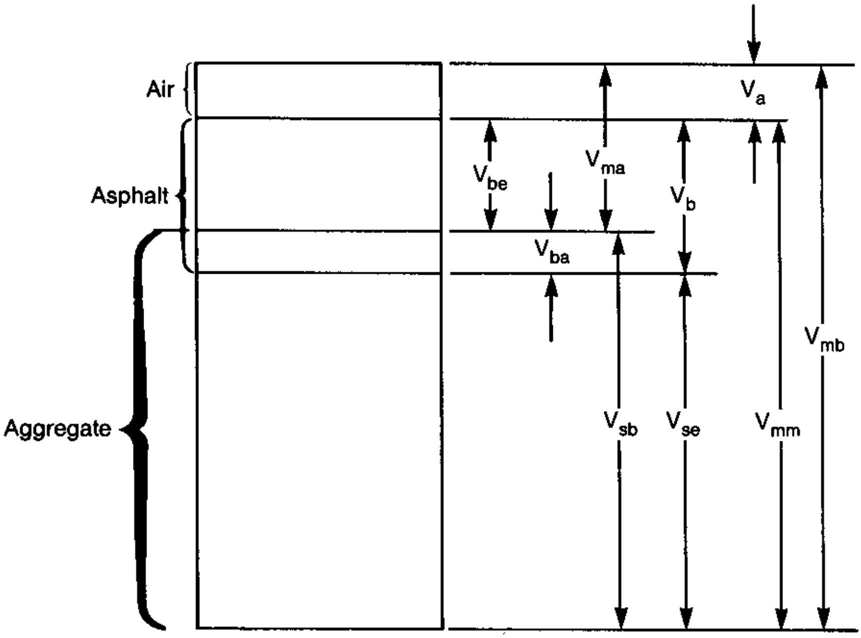
Density-Voids Analysis

Three properties of the compacted asphalt paving mixture specimens are determined in the density-voids analysis. They are the following:

- density and/or specific gravity of the compacted specimen
- voids in the mineral aggregate
- air voids in the compacted mix

The density of the compacted specimen in mix design testing and analysis is usually expressed in g/cm^3 (lb/ft^3). The bulk specific gravity of the specimen multiplied by the density of water, 1.0 g/cm^3 (62.4 lb/ft^3) gives the density of the compacted paving sample.

From the sample weight and percentages of asphalt and aggregate and specific gravities of each, the volumes of the respective materials may be determined. These volumes are shown in Figure 4.1. The bulk volume is the volume of water displaced by the compacted sample.



V_{ma} = Volume of voids in mineral aggregate
V_{mb} = Volume of compacted specimen
V_{mm} = Voidless volume of paving mix
V_a = Volume of air voids
V_b = Volume of asphalt
V_{ba} = Volume of absorbed asphalt
V_{be} = Volume of effective asphalt
V_{sb} = Volume of aggregate (by bulk specific gravity)
V_{se} = Volume of aggregate (by effective specific gravity)
W_b = Weight of asphalt
W_s = Weight of aggregate
γ_w = Unit weight of water 1.0 g/cm^3 (62.4 lb/ft^3)
G_{mb} = Bulk specific gravity of compacted paving mixture sample

$$\text{Density of compacted specimen} = \left(\frac{W_b + W_s}{V_{mb}}\right) \text{X} \ \gamma_w = G_{mb} \ \gamma_w$$

$$\text{Percent voids in mineral aggregate} = \left(\frac{V_{be} + V_a}{V_{mb}}\right) 100$$

$$\text{Percent voids in compacted specimen} = \left(\frac{V_a}{V_{mb}}\right) 100$$

Figure 4.1 Density-Voids and Volume Relationships of a Compacted Specimen of Asphalt Paving Mix.

Voids in mineral aggregate (VMA) are expressed as a percent of the total volume of the specimen. They represent that volume of the compacted mix that is not occupied by aggregate and absorbed asphalt.

Voids in the mix, V_a, are also expressed as a percent of the total volume of the specimen. They represent that volume occupied by neither asphalt nor aggregate. Unless the amount of asphalt absorbed by the aggregate is taken into account, the calculated percent of air voids in the mix may be deceptively low.

Evaluating and Adjusting Mix Designs

In the process of developing a specific mix design, often it is necessary to make several trial mixes in order to satisfy the design criteria of the particular design method in use. Where the trial mixes fail to meet the design criteria, it is necessary to modify or, in some cases, redesign the mix. Certain adjustments in the gradation of the aggregate blend, within the specification limits, may be required to correct the deficiency.

In evaluating and adjusting mix designs, the engineer must always keep in mind that the aggregate gradation and asphalt content in the final mix must strike a favorable balance between stability and durability requirements. The aim should be for adequate stability and maximum durability.

4.2 Procedure for Analyzing a Compacted Paving Mixture

Introduction

The following analytical procedures apply to paving mixtures compacted in a laboratory and to undisturbed samples cut from compacted pavement.

By analyzing a compacted paving mixture for air voids and voids in the mineral aggregate (VMA), some indication of its probable durability and in-service performance may be determined. Air voids are the small air spaces between coated aggregate particles; VMA consists of the intergranular void spaces between the particles of aggregate that hold asphalt in a compacted mixture. They are expressed as percent by volume of the total volume of the compacted mixture.

It is possible to develop many equations for use in density-voids analyses. When the basic concepts are understood, the development of new equations to fit a particular set of measurements is a relatively simple procedure.

Information regarding the ingredients of an asphalt paving mixture must be known before any weight-volume calculations are made. It includes the specific gravity of the asphalt, the bulk specific gravity for each aggregate and the proportions of each ingredient in the mix.

In addition, a set of weight measurements must be made on an uncompacted sample of mix for determining the amount of asphalt absorbed by the aggregate. With this information, it is possible to make a density-voids analysis of compacted asphalt paving specimens whose weight and bulk volume are known.

To aid in understanding the fundamentals involved, a block diagram of asphalt paving mixture is shown in Figure 4.1. The properties are shown in terms of volume, but they may be expressed in weights also, using the same subscripts. Figure 4.1 should be consistently referred to as equations are developed.

Data for all illustrative calculations in this section are based on the information given in Table 4.1. All of the density-voids calculations involve measurements of the materials at 25°C (77°F), including specific gravity. In some cases submerged weights are necessary. Even though the density of water is 0.997 g/cm^3 (62.26 lb/ft^3) at 25°C (77°F), it is assumed to be 1 g/cm^3 (62.4 lb/ft^3). The error induced by this assumption is considered insignificant.

Procedure Outline

The following is an outline of steps to follow in the analysis of paving mixtures:

(1) Measure the bulk specific gravities of the coarse aggregate (AASHTO T 85 or ASTM C 127) and of the fine aggregate (AASHTO T 84 or ASTM C 128).

(2) Measure the specific gravity of the asphalt cement (AASHTO T 228 or ASTM D 70) and of the mineral filler (AASHTO T 100 or ASTM D 854).

(3) Calculate the bulk specific gravity of the aggregate combination in the paving mixture.

(4) Measure the maximum specific gravity of the loose paving mixture (ASTM D 2041).

(5) Measure the bulk specific gravity of the compacted paving mixture (ASTM D 1188 or ASTM D 2726).

(6) Calculate the effective specific gravity of the aggregate.

(7) Calculate asphalt absorption of the aggregate.

(8) Calculate the effective asphalt content of the paving mixture.

(9) Calculate the percent voids in the mineral aggregate in the compacted paving mixture.

(10) Calculate the percent air voids in the compacted paving mixture.

Equations for the above calculations are found in the discussions that follow.

Paving Mixture Data for Sample Calculations

Table 4.1 illustrates the basic data for a sample of paving mixture. These design data are used in the sample calculation employed throughout the remainder of the chapter.

Table 4.1 Basic Data for Sample of Paving Mixture

(a) Constituents

	Specific Gravity				*Mix Composition*	
Material		**Bulk**	**AASHTO Method**	**ASTM Method**	**Percent By Weight of Total Mix**	**Percent By Weight of Total Aggregate**
Asphalt Cement	1.010 (G_b)	—	T 228	D 70	6.96 (P_b)	7.48 (P_b)
Coarse Aggregate		2.606 (G_1)	T 85	C 127	51.45 (P_1)	55.30 (P_1)
Fine Aggregate		2.711 (G_2)	T 84	C 128	34.24 (P_2)	36.80 (P_2)
Mineral Filler	2.697 (G_3)	—	T 100	D 854	7.35 (P_3)	7.90 (P_3)

(b) Paving Mixture

Bulk specific gravity of compacted paving mixture sample, G_{mb}

(ASTM D 2726) ______ 2.344

Maximum specific gravity of paving mixture sample, G_{mm}

(ASTM D 2041) ______ 2.438

Bulk Specific Gravity of Aggregate

When the total aggregate consists of separate fractions of coarse aggregate, fine aggregate, and mineral filler, all having different specific gravities, the bulk specific gravity for the aggregate combination is calculated as follows:

$$G_{sb} = \frac{P_1 + P_2 + \ldots + P_n}{\frac{P_1}{G_1} + \frac{P_2}{G_2} + \ldots + \frac{P_n}{G_n}} \qquad (1)$$

where

G_{sb} = bulk specific gravity for the total aggregate
P_1, P_2, P_n = percentages by weight of aggregates 1, 2, n; and
G_1, G_2, G_n = bulk specific gravities of aggregates 1, 2, n.

The bulk specific gravity of mineral filler is difficult to determine accurately at the present time. However, if the apparent specific gravity of the filler is used instead, the error is usually negligible.

Calculation using the data in Table 4.1:

$$G_{sb} = \frac{51.450 + 34.240 + 7.350}{\frac{51.450}{2.606} + \frac{34.240}{2.711} + \frac{7.350}{2.697}} = \frac{93.040}{19.743 + 12.630 + 2.725} = 2.651$$

Effective Specific Gravity of Aggregate

When based on the maximum specific gravity of a paving mixture, G_{mm}, the effective specific gravity of G_{se}, of the aggregate includes all void spaces in the aggregate particles except those that absorb asphalt. It is determined as follows:

$$G_{se} = \frac{P_{mm} - P_b}{\frac{P_{mm}}{G_{mm}} - \frac{P_b}{G_b}} \qquad (2)$$

where

G_{se} = effective specific gravity of aggregate
P_{mm} = total loose mixture, percent by total weight of mixture = 100 percent
P_b = asphalt, percent by total weight of mixture
G_{mm} = maximum specific gravity of paving mixture (no air voids), ASTM D 2041
G_b = specific gravity of asphalt

Calculation using the data in Table 4.1

$$G_{se} = \frac{100 - 6.960}{\dfrac{100}{2.438} - \dfrac{6.960}{1.010}} = \frac{93.040}{34.126} = 2.726$$

NOTE:

The volume of asphalt binder absorbed by an aggregate is almost invariably less than the volume of water absorbed. Consequently, the value for the effective specific gravity of an aggregate should be between its bulk and apparent specific gravities. When the effective specific gravity falls outside these limits, its value must be assumed to be incorrect. The calculations, the maximum specific gravity of the total mix by ASTM D 2041, and the composition of the mix in terms of aggregate and total asphalt content should then be rechecked for the source of the error.

If the apparent specific gravity of the coarse aggregate is 2.759 and the apparent specific gravity of the fine aggregate is 2.905 for the sample of paving mix in Table 4.1, the apparent specific gravity, G_{sa}, of the total aggregate can be calculated by the same formula as the bulk by using the apparent specific gravity of each aggregate constituent. For this example, then, the calculated apparent specific gravity, G_{sa}, is

$$G_{sa} = \frac{51.450 + 34.240 + 7.350}{\dfrac{51.450}{2.759} + \dfrac{34.240}{2.905} + \dfrac{7.350}{2.697}} = \frac{93.040}{33.160} = 2.806$$

In the example the three specific gravities are as follows:

Bulk Specific Gravity, G_{sb}	2.651
Effective Specific Gravity, G_{se}	2.726
Apparent Specific Gravity, G_{sa}	2.806

Maximum Specific Gravities of Mixtures With Different Asphalt Contents

In designing a paving mixture with a given aggregate, the maximum specific gravities, G_{mm}, at different asphalt contents are needed to calculate the percentage of air voids for each asphalt content. While the same maximum specific gravity can be determined for each asphalt content by ASTM test method D 2041, the precision of the test is best when the mixture has close to the optimum asphalt content. Also, it is preferable to measure the maximum specific gravity in duplicate or triplicate.

After averaging the results from these tests and calculating the effective specific gravity of the aggregate, the maximum specific gravity for any other asphalt content can be obtained as shown below. For all practical purposes, the effective specific gravity of the aggregate is constant because the asphalt absorption does not vary appreciably with variations in asphalt content.

$$G_{mm} = \frac{P_{mm}}{\dfrac{P_s}{G_{se}} + \dfrac{P_b}{G_b}} \tag{3}$$

where

G_{mm} = maximum specific gravity of paving mixture (no air voids)
P_{mm} = total loose mixture, percent by total weight of mixture = 100 percent
P_s = aggregate, percent by total weight of mixture
P_b = asphalt, percent by total weight of mixture
G_{se} = effective specific gravity of aggregate
G_b = specific gravity of asphalt

Calculations using specific gravity data from Table 4.1, effective specific gravity, G_{se}, determined on the previous page and on asphalt content, P_b, of 7.46 percent:

$$G_{mm} = \frac{100}{\dfrac{92.540}{2.726} + \dfrac{7.460}{1.010}} = \frac{100}{41.333} = 2.419$$

Asphalt Absorption

Absorption is expressed as a percentage by weight of aggregate rather than as a percentage by total weight of mixture. Asphalt asborption (P_{ba}) is determined as follows:

$$P_{ba} = 100\left(\frac{G_{se} - G_{sb}}{G_{sb}\ G_{se}}\right) G_b \tag{4}$$

where

P_{ba} = absorbed asphalt, percent by weight of aggregate
G_{se} = effective specific gravity of aggregate
G_{sb} = bulk specific gravity of aggregate
G_b = specific gravity of asphalt

Calculation using specific gravity data from Table 4.1

$$P_{ba} = 100\frac{2.726 - 2.651}{2.651 \times 2.726} 1.010 = 100 \frac{0.075}{7.227} 1.010 = 1.05$$

Effective Asphalt Content of a Paving Mixture

The effective asphalt content (P_{be}) of a paving mixture is the total asphalt content minus the quantity of asphalt lost by absorption into the aggregate particles. It is the portion of the total asphalt content that remains as a coating on the outside of the aggregate particles, and is the asphalt content on which service performance of an asphalt paving mixture depends. The formula is:

$$P_{be} = P_b - \left(\frac{P_{ba}}{100}\right) P_s \tag{5}$$

where

P_{be} = effective asphalt content, percent by total weight of mixture
P_b = asphalt, percent by total weight of mixture
P_{ba} = absorbed asphalt, percent by weight of aggregate
P_s = aggregate, percent by total weight of mixture

The following is an example using data calculated in the previous examples.

$$P_{be} = 6.96 - \frac{1.05}{100} 93.04 = 6.96 - 0.98 = 5.98$$

Percent VMA in Compacted Paving Mixture

The voids in the mineral aggregate, VMA, are defined as the intergranular void space between the aggregate particles in a compacted paving mixture that includes the air voids and the effective asphalt content, expressed as a percent of the total volume. The VMA is calculated on the basis of the bulk specific gravity of the aggregate and is expressed as a percentage of the bulk volume of the compacted paving mixture. Therefore, the VMA can be calculated by subtracting the volume of the aggregate determined by its bulk specific gravity from the bulk volume of the compacted paving mixture. The method of calculation is illustrated as follows:

If mix composition is determined as percent by weight of total mixture:

$$\mathrm{VMA} = 100 - \left(\frac{G_{mb}\,P_s}{G_{sb}}\right) \tag{6}$$

where

VMA = voids in mineral aggregate (percent of bulk volume)
G_{sb} = bulk specific gravity of aggregate
G_{mb} = bulk specific gravity of compacted mixture (ASTM D 2726)
P_s = aggregate, percent by total weight of mixture

Calculation using data from Table 4.1

$$\mathrm{VMA} = 100 - \frac{2.344 \times 93.04}{2.651} = 100 - 82.27 = 17.73$$

Or if mix composition is determined as percent by weight of aggregate:

$$\mathrm{VMA} = 100 - \frac{G_{mb}}{G_{sb}} \times \frac{100}{100 + P_b} 100 \tag{7}$$

where

P_b = asphalt, percent by weight of aggregate

Calculation using data from Table 4.1

$$\mathrm{VMA} = 100 - \frac{2.344}{2.651} + \frac{4.6}{100 + 7.48} 100 = 100 - 82.27 = 17.73$$

Calculation of Air Voids in Compacted Mixture

The air voids, P_a, in a compacted paving mixture consist of the small air spaces *between* the coated aggregate particles. The percentage of air voids in a compacted mixture can be determined by the following equation:

$$P_a = 100 \frac{G_{mm} - G_{mb}}{G_{mm}} \tag{8}$$

where

P_a = air voids in compacted mixture, percent of total volume
G_{mm} = maximum specific gravity of paving mixture (as determined in previous discussion, or as determined directly for a paving mixture by ASTM Method D 2041)
G_{mb} = bulk specific gravity of compacted mixture

Calculation using data from Table 4.1.

$$P_a = 100 \frac{2.438 - 2.344}{2.438} = \frac{9.400}{2.438} = 3.86$$

4.3 Asphalt Hot Mix Design Methods

Introduction

Design Method and Requirements

Mix design methods and design requirements form an essential part of construction specifications for all heavy-duty asphalt paving. Regardless of the mix design used, it becomes a part of the specifications and not an "authority" over and above the specifications.

The agency or authority responsible for paving construction usually establishes the mix design method and design requirements. Once these are established, it becomes the responsibility of the engineer and technician to develop the mix design within the framework of the other specification requirements.

The Marshall and Hveem methods of mix design have been widely used, with satisfactory results. For each method, criteria have been developed by correlating the results of laboratory tests on compacted paving mixes with the performance of the paving mixes under service conditions. In every instance, however, the correlation was made within certain limits; these limits are presented in this section for each method. Each design method is not applicable for all types of mixes. Suitability of laboratory design methods for different types of mixes is given in Table 4.2.

Table 4.2 Suitability of Laboratory Design Methods

ASTM Paving Mix Designation*	Marshall	Hveem
37.5 mm (1½ in.)	U	U
25.0 mm (1 in.)	O	O
19.0 mm (¾ in.)	✓	✓
12.5 mm (½ in.)	✓	✓
9.5 mm (⅜ in.)	✓	✓
4.75 mm (No. 4)	✓	✓
2.36 mm (No. 8)	✓	✓
1.18 mm (No. 16)	✓	✓

✓ = Suitable
o = Suitable if maximum size aggregate is 25 mm (1 in.) or less.
u = Unsuitable, mix design modifications are required.

*ASTM D 3515, Table 1.

Schedule of Samples and Tests

Prior to mix design testing, the engineer/inspector should determine to his own satisfaction whether representative samples of aggregates and asphalt are in ample supply to accomplish the required number of tests. The following schedule of quantities is suggested:

- 4 litres (1 gal) asphalt cement
- 23 kg (50 lb) coarse aggregate (or rock)
- 23 kg (50 lb) fine aggregate (or sand)
- 9 kg (20 lb) mineral filler (when required)

Additional quantities of aggregate may be required if these quantities result in appreciable waste in combining materials for the design gradation, or if several combinations of coarse and fine aggregates are being investigated.

Each sample of material should be identified by source location, project location, and project or job number. Asphalt cement samples should be in clean, small metal containers with tight lids or covers to prevent reheating the entire supply each time a mix is tested. Individual aggregate samples should be in a tightly woven cloth sack, securely wired or tied.

A list or schedule of tests to be performed should be compiled by the inspector. Tests should be listed in their proper and logical sequence. It is important that all other specification tests be completed before the mix design tests are started. In this way sources of substandard aggregates are eliminated from the design studies.

Preparation of Test Mixes

General procedures for preparing test mixes are illustrated in Figures 4.2 to 4.5. The procedure generally consists of:

1. Drying aggregates to constant weight,
2. Dry sieving aggregates into desired size fractions,
3. Weighing aggregates for batch mixes,
4. Heating aggregate batch mixes in the oven,
5. Placing aggregate batch mixes in mixing bowl,
6. Adding prescribed amounts of hot asphalt to batch, and
7. Mixing aggregate and asphalt together.

Marshall Method of Mix Design

Development and Application

The concepts of the Marshall method of designing paving mixtures were formulated by Bruce Marshall, formerly Bituminous Engineer with the Mississippi State Highway Department. The U.S. Corps of Engineers, through extensive research and correlation studies, improved and added certain features to Marshall's test procedure, and ultimately developed mix design criteria. The Marshall test procedures have been standardized by the

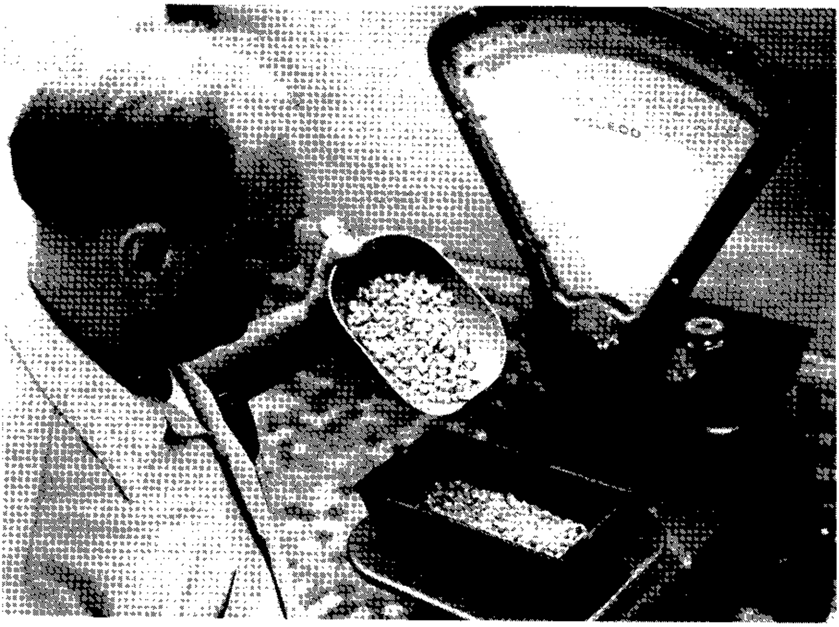


Figure 4.2 Weighing Aggregates for Batch Mixes.

Figure 4.3 Heating Aggregate Batches in Oven.

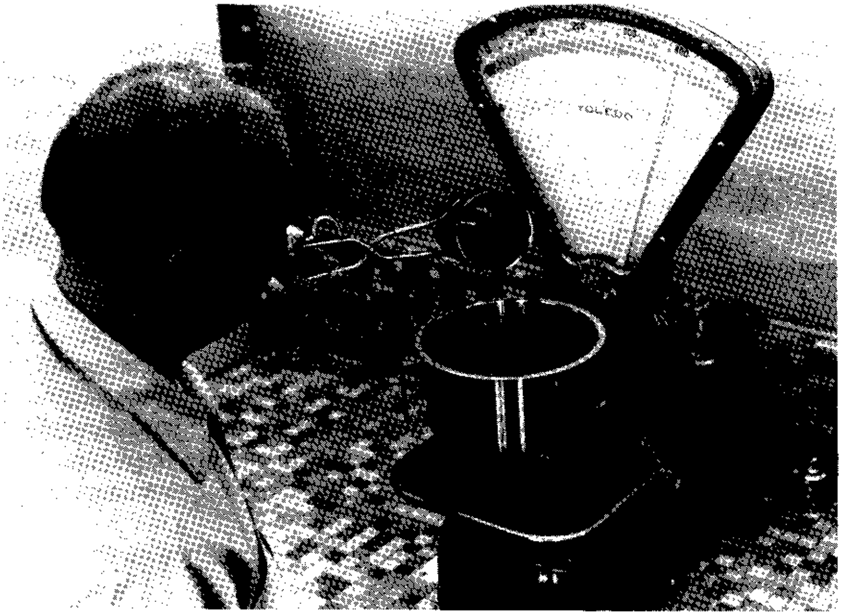


Figure 4.4 Adding Asphalt From a Thermostatically-Controlled Asphalt Heating Kettle.

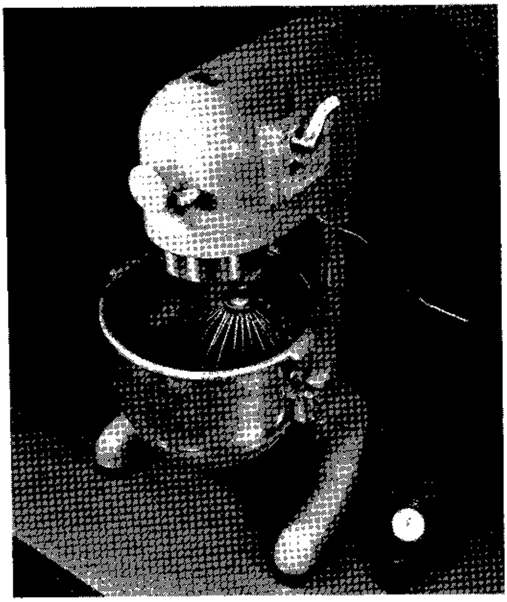


Figure 4.5 Mechanical Mixer for Batch Mixing of Asphalt and Aggregate.

American Society for Testing and Materials. Procedures given by ASTM Designation D 1559, *Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus.** Testing procedures presented here are basically the same as those of the ASTM Method. (It is recommended that where agencies reference either AASHTO or ASTM, that the particular procedure be exactly followed.)

The Marshall method as presented here is applicable only to hot-mix asphalt paving mixtures using penetration or viscosity graded asphalt cements and containing aggregates with maximum size of 25 mm (1 in.) or less. The method is intended for laboratory design of asphalt hot-mix paving.

Outline of Method

The procedure for the Marshall method starts with the preparation of test specimens. Preliminary to this operation the following are required:

(1) That the materials proposed for use meet the requirements of the project specifications;

(2) That aggregate blend combinations meet the gradation requirements of the project specifications; and

(3) That, for use in density and voids analyses, the bulk specific gravity of all aggregates used in the blend, and the specific gravity of the asphalt cement, are determined.

These requirements are matters of routine testing, specifications, and laboratory technique that must be considered but that are not unique to any particular mix design method.

The Marshall method uses standard cylindrical test specimens of 64mm (2½ in.) height × 102 mm (4 in.) diameter. These are prepared using a specified procedure for heating, mixing, and compacting the asphalt-aggregate mixtures. The two principal features of the Marshall method of mix design are a density-voids analysis and a stability-flow test of the compacted test specimens.

The stability of the test specimens is the maximum load resistance in Newtons (lb.) that the standard test specimen will develop at 60° C (140° F) when tested as outlined hereinafter. The flow value is the total movement or strain, in units of 0.25 millimetre (1/100 in), occurring in the specimen between no load and maximum load during the stability test.

Preparation of Test Specimens

In determining the optimum asphalt content for a particular blend or gradation of aggregates by the Marshall method, a series of test specimens is prepared for a range of different asphalt contents so that the test data curves show a well-defined "optimum" value. Tests should be scheduled on the basis of ½ percent increments of asphalt content, with at least two asphalt contents above "optimum" and at least two below "optimum."

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*AASHTO T 245 "Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus" agrees with ASTM D 1559 except for provisions for mechanically-operated hammer. AASHTO T 245 Par. 2.3 Note 2—Instead of a hand-operated hammer and associated equipment, a mechanically-operated hammer may be used provided it has been calibrated to give results comparable to the hand-operated hammer.

The equipment required for the preparation of test specimens is as follows:

Pans, metal, flat bottom, for heating aggregates.

Pans, metal, round, approximately 4-litres (4-qt.) capacity, for mixing asphalt and aggregate.

Oven and Hot Plate, electric, for heating aggregates, asphalt and equipment as required.

Scoop, for batching aggregates.

Containers, gill-type tins, beakers, pouring pots, or sauce pans, for heating asphalt.

Thermometers, armored, glass, dial-type with metal stem, or electronic 10° C (50° F) to 232° C (450° F), for determining temperature of aggregates, asphalt and asphalt mixtures.

Balance, 5-kg capacity, sensitive to 1 gm for weighing aggregates and asphalt. *Balance,* 2-kg capacity, sensitive to 0.1 gm for weighing compacted specimens.

Mixing Spoon, large, or *Trowel,* small.

Spatula, large.

Mechanical Mixer (optional), commercial bread dough mixer 4-litre (4 qt.) capacity or larger, equipped with two metal mixing bowls and two wire stirrers.

Boiling Water Bath, consisting of hot plate and bucket for water, for heating compaction hammer and mold.

Compaction Pedestal, Figure 4.6, consisting of a 200 × 200 × 460mm (8 × 8 × 18 in.) wooden post capped with a 305 × 305 × 25 mm (12 × 12 × 1 in.) steel plate. The wooden post should be oak, yellow pine or other wood having a dry weight of 673 to 769 kg/m^3 (42 to 48 lb./ft^3). The wooden post should be secured by four angle brackets to a solid concrete slab. The steel cap should be firmly fastened to the post. The pedestal should be installed so that the post is plumb, the cap level, and the entire assembly free from movement during compaction.

Compaction Mold,* consisting of a base plate, forming mold and collar extension. The forming mold has an inside diameter of 101.6 mm (4 in.) and a height of approximately 75 mm (3 in.); the base plate and collar extension are designed to be interchangeable with either end of the forming mold.

Compaction Hammer,* consisting of a flat circular tamping face 98.4 mm (3 ⅞ in.) diameter and equipped with a 4.5 kg (10 lb.) weight constructed to obtain a specified 457 mm (18 in.) height of drop.

Mold Holder,* consisting of spring tension device designed to hold compaction mold in place on compaction pedestal.

*Extrusion Jack** or Arbor Press, for extruding compacted specimens from mold.

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*Marshall test apparatus should conform to requirements of ASTM test method D 1559.

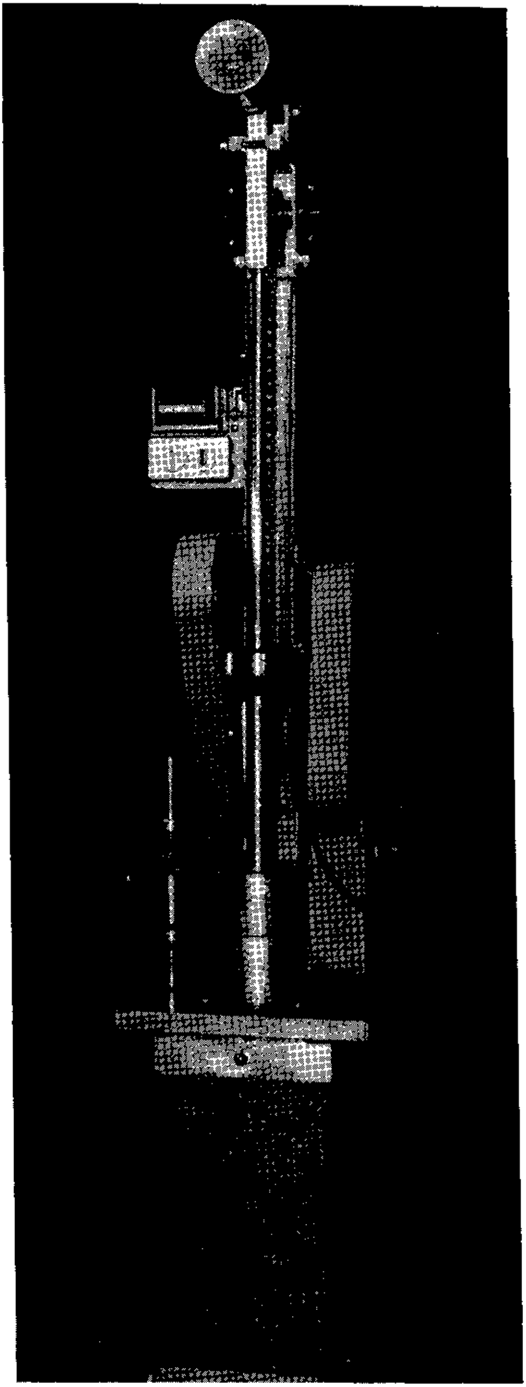


Figure 4.6 Mechanical Hammer Used in Preparing Marshall Test Specimens.

Gloves, welders, for handling hot equipment. *Gloves,* rubber, for removing specimens from water bath.

Marking Crayons, for identifying test specimens.

To provide adequate data, *triplicate* test specimens are the minimum prepared for each asphalt content used. Thus, a hot-mix design study using six different asphalt contents will normally require at least eighteen test specimens. Each test specimen will usually require approximately 1.2 kg (2 ¼ lb) of aggregate. Therefore, the *minimum* aggregate requirements for one series of test specimens of a given blend and gradation will be approximately 23 kg (50 lb.) Four litres (one gallon) approximately of asphalt cement will be adequate. The following steps are to be followed in preparing specimens.

(a) *Number of Specimens*—Prepare at least three, and preferably five, specimens for each combination of aggregates and asphalt content.

(b) *Preparation of Aggregates*—Dry aggregates to constant weight at 105°C (221°F) to 110°C (230°F) and separate the aggregates by dry-sieving into the desired size fraction. The following size fractions are recommended as generally adequate: 25.0 to 19.0 mm (1 to ¾ in.), 19.0 to 9.5 mm (¾ to ⅜ in.), 9.5 to 4.75 mm (⅜ in. to No. 4), 4.75 to 2.36 mm (No. 4 to No. 8), passing 2.36 mm (No. 8).

(c) *Determination of Mixing and Compaction Temperature*—The temperature to which the asphalt must be heated to produce viscosities of 170 ± 20 centistokes kinematic and 280 ± 30 centistokes kinematic shall be established as the mixing temperature and compaction temperature, respectively.

(d) *Preparation of Mold and Hammer*—Thoroughly clean the specimen mold assembly and the face of the compaction hammer and heat them in a boiling water bath or on the hot plate to a temperature between 93° C (200° F) and 149° C (300° F). Place a piece of waxed paper cut to size in the bottom of the mold before the mixture is placed in the mold.

(e) *Preparation of Mixtures*—Weigh into separate pans for each test specimen the amount of each size fraction required to produce a batch that will result in a compacted specimen 63.5 ± 1.3 mm (2.5 ± 0.05 in.) in height. This will normally be about 1.2 kg. It is generally desirable to prepare a trial specimen prior to preparing the aggregate batches. If the trial specimen height falls outside the limits, the amount of aggregate used for the specimen may be adjusted as follows:

For International System of Units (SI),

$$\text{Adjusted mass of aggregate} = \frac{63.5\ (\text{mass of aggregate used})}{\text{Specimen height (mm) obtained}}$$

or

U.S. Customary Units,

$$\text{Adjusted weight of aggregate} = \frac{2.5\ (\text{weight of aggregate used})}{\text{Specimen height (in.) obtained}}$$

Place the pans in the oven or on the hot plate and heat to a temperature approximately 28° C (50° F) above the mixing temperature specified in (c). If a hot plate is used, provision should be made for dead space, baffle plate, or a sand bath beneath the pans and the hot plate to prevent local overheating. Charge the mixing bowl with heated aggregates and dry mix thoroughly. Form a crater in the dry blended aggregate and weigh the required amount of asphalt cement into the mixture in accordance with the accumulative batch weights. At this point the temperature of the aggregate and asphalt must be within the limits of the mixing temperature established in paragraph (c). Asphalt should not be held at mixing temperatures for more than one hour before using. Mix the aggregate and asphalt cement, preferably with a mechanical mixer or by hand with a trowel, as quickly and thoroughly as possible to yield a mixture having a uniform distribution of asphalt throughout.

(f) *Compaction of Specimens*—Place the entire batch in the mold, spade the mixture vigorously with a heated spatula or trowel 15 times around the perimeter and ten times over the interior. Remove the collar and smooth the surface to a slightly rounded shape. The temperature of the mixture immediately prior to compaction shall be within the limits of the compaction temperature established in paragraph (c); otherwise, it shall be discarded. In no case shall the mixture be reheated.

(g) Replace the collar, place the mold assembly on the compaction pedestal in the mold holder. Apply either 35, 50, or 75 blows, as specified according to the design traffic category, with the compaction hammer using a free fall of 457 mm (18 in.). Hold the axis of the compaction hammer as nearly perpendicular to the base of the mold assembly as possible during compaction. Remove the base plate and collar, and reverse and reassemble the mold. Apply the same number of compaction blows to the face of the reversed specimen. A mechanical compactor may be used but the relationship between hand and mechanical tamping should be established. After compaction, remove the base plate and allow the specimen to cool in air until no deformation will result when removing it from the mold.

When more rapid cooling is desired table fans may be used, but not water unless the specimen is in a plastic bag. Remove the specimen from the mold by means of an extrusion jack or other compression device, then place on a smooth, level surface until ready for testing. Normally, specimens are allowed to cool overnight.

Mix Design Test Procedure

In the Marshall method each compacted test specimen is subjected to the following tests and analysis in the order listed:

- Bulk Specific Gravity Determination
- Stability and Flow Test
- Density and Voids Analysis

The equipment required for the testing of the 102 mm (4 in.) diameter × 64 mm (2 ½ in.) height specimens consists of a compression testing device and a water bath. The compression testing device can be any mechanical or hydraulic testing machine (Figure 4.7) equipped with suitable load and deformation indicating devices provided the capacity and sensitivity meet the requirements outlined in ASTM D 1559. Alternatively, a test press as shown in Figure 4.8 may be used to conduct the stability and flow tests. The water bath should be at least 150 mm (6 in.) deep and thermostatically controlled to 60° C ± 1° C (140° F ± 1.8° F). The tank should have a perforated false bottom or be equipped with a shelf for suspending specimens at least 50 mm (2 in.) above the bottom of the bath.

(a) *Bulk Specific Gravity Determination*

The bulk specific gravity test may be performed as soon as the freshly compacted specimens have cooled to room temperature. This test is performed according to ASTM Designation D 1188 *Bulk Specific Gravity of Compacted Bituminous Mixtures Using Paraffin-Coated Specimens or* ASTM D 2726 *Bulk Specific Gravity of Compacted Bituminous Mixtures Using Saturated Surface-Dry Specimens.*

(b) *Stability and Flow Tests*

After the bulk specific gravity of the test specimens has been determined, the stability and flow tests are performed utilizing a compression testing device as shown in Figure 4.7 or 4.8. The following steps are applicable to the test press in Figure 4.7.

1. "Zero" the flow meter by inserting a 101.6 mm (4.00 in.) diameter metal cylnder in the testing head, placing the flow meter over the guide rod and adjusting the flow meter to read "zero."

(Note: This adjustment should be made on the guide post marked with an "O" and with the side of the upper segment of the testing head marked with an "O" being placed on the same side as the guide post so marked. The same assembly of testing head and flow meter must then be used in testing the specimens. Specimens should be 101.6 ± 0.25 mm [400 mm ± 0.01 in.]; otherwise, an initial and final reading of flow meter is required for the determination of the flow value.)

2. Immerse specimen in water bath at 60° C ± 1° C (140° F ± 1.8° F) for 30 to 40 minutes before test.

3. Thoroughly clean inside surfaces of testing head. Temperature of head shall be maintained at from 21° to 37.8° C (70° to 100° F) using a water bath when required. Lubricate guide rods with a thin film of oil so that upper test head will slide freely without binding. If a proving ring is used to measure applied load, check to see that dial indicator is firmly fixed and "zeroed" for the "no-load" position.

(4) With testing apparatus in readiness, remove test specimen from water bath and carefully dry surface. Place specimen in lower testing head and center; then fit upper testing head into position and center complete assembly in loading device. Place flow meter over marked guide rod as noted in (1) above.

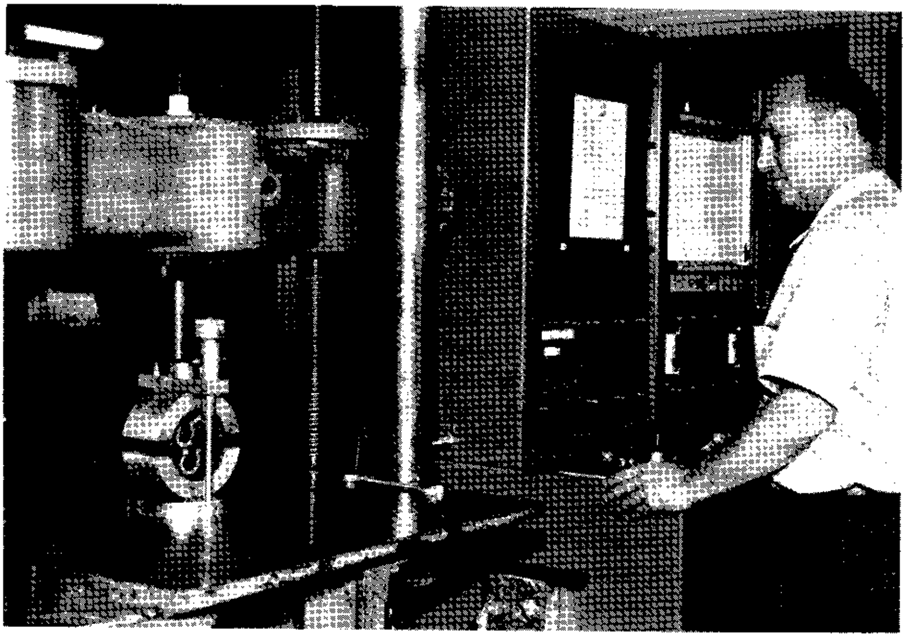


Figure 4.7 **Marshall Procedure, Compression Testing Device.**

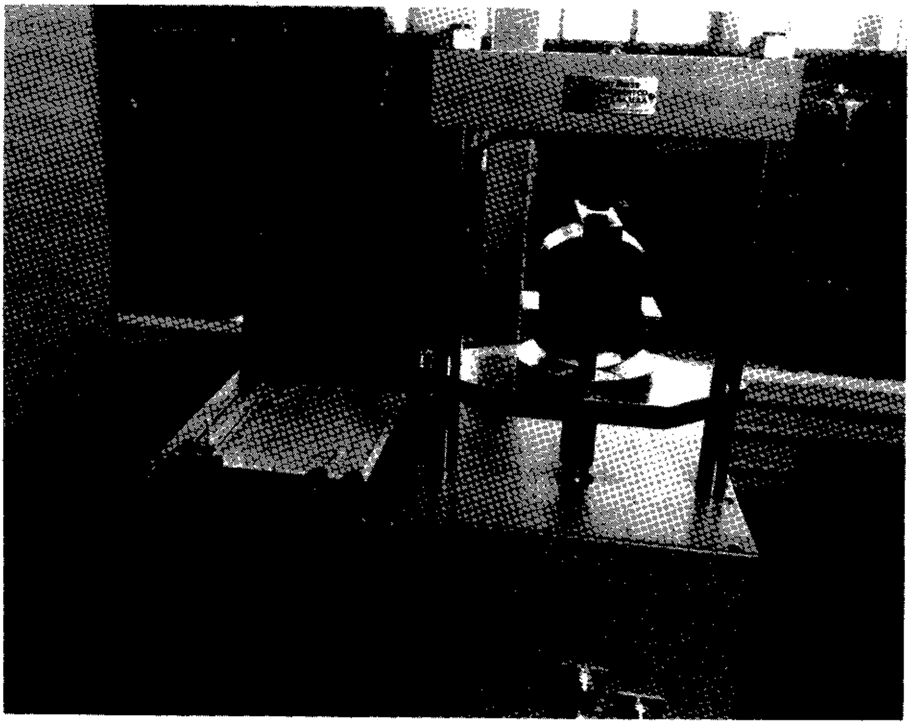


Figure 4.8 **Test Press.** *(Courtesy Pine Instrument Co.)*

(5) Apply testing load to specimen at constant rate of deformation, 51 mm (2 in.) per minute, until failure occurs. The point of failure is defined by the maximum load reading obtained. The total number of Newtons (lb.) required to produce failure of the specimen at 60° C (140° F) shall be recorded as its Marshall stability value.

(6) While stability test is in progress, hold the flow meter firmly in position over guide rod and remove as the load begins to decrease; take reading and record. This reading is the flow value for the specimen, expressed in units of 0.25 mm (1/100 in.). For example, if the specimen deformed 3.8 mm (0.15 in.) the flow value is 15.

(7) The entire procedure, both stability and flow tests, starting with the removal of the specimen from the water bath, shall be completed within a period of thirty seconds.

(c) *Density - Voids Analysis*

After completion of the stability and flow tests a density and voids analysis is made for each series of test specimens as outlined in the previous discussion and shown in Figure 4.9.

Interpretation of Test Data

The stability and flow values and void data are prepared as follows:

(1) Measured stability values for specimens that depart from the standard 63.5 mm (2½ in.) thickness shall be converted to an equivalent 63.5 mm (2½ in.) value by means of a conversion factor. Applicable correlation ratios to convert the measured stability values are set forth in Table 4.3. Note that the conversion may be made on the basis of either measured thickness or measured volume.

(2) Average the flow values and the converted stability values for all specimens of a given asphalt content. Values that are obviously in error shall not be included in the average.

(3) Prepare a separate graphical plot for the following values as illustrated in Figure 4.10.

- Stability vs. Asphalt Content
- Flow vs. Asphalt Content
- Unit Weight of Total Mix vs. Asphalt Content
- Percent Air Voids vs. Asphalt Content
- Percent Voids in Mineral Aggregate (VMA) vs. Asphalt Content.

In each case connect the plotted values with a smooth curve that obtains the "best-fit" for all values.

The test property curves, Figure 4.10, have been found to follow a reasonably consistent pattern for dense-graded asphalt paving mixes. Trends generally noted are outlined as follows:

- The stability value increases with increasing asphalt content up to a maximum after which the stability decreases.
- The flow value increases with increasing asphalt content.

HOT MIX DESIGN DATA
by the
MARSHALL METHOD

Trial Mix Series: 3 - B
52% C.A.; 48% F.A.
75 Blow Compaction

Project: F-003-4(2)
Location: Lewis - North
Date: Jan. 6, 1984

Sp. Gr. AC: 1.022
Avg. Bulk Sp. Gr. Total Agg. 2.885

Pen. Grade AC: 85-100
Absorbed AC, lbs./100 lbs. + Dry Agg. 0.16

Lab. No. for AC Used: 83-205
Lab. Nos. for Agg. Used: 83-551, 83-552, 83-553

% AC SPEC. NO.	% AC SPEC. NO.	SPEC. HGT. mm (in.)	MASS GRAMS			BULK VOL. CC.	BULK S.G. COMP. MIX	AIR VOIDS	% V.M.A.	UNIT WGT. PCF. (Mg/m³)*	STABIL -LBS (N)**		FLOW 1/100" (0.25 mm)
			IN AIR	IN WATER	SAT. SURFACE DRY IN AIR						MEAS.	ADJUST. †	
% AC by wgt. of Agg.	% AC by wgt. of Mix												
4.17	4.0-A		1306.8	798.6	1310.6	512.0	2.552				1790	1790	8
	B		1309.5	800.7	1313.8	513.1	2.552				1810	1810	8
	C		1309.6	803.0	1314.1	511.1	2.562				1850	1850	8
	Avg.						2.555	5.2	15.0	159.4		1817	8
4.71	4.5-A		1315.3	808.5	1317.5	509.0	2.584				1900	1900	8
	B		1315.1	808.9	1317.2	508.3	2.587				1735	1804	10
	C		1311.6	801.8	1309.9	508.1	2.581				1810	1882	8
	Avg.						2.584	3.5	14.5	161.2		1862	9
5.26	5.0-A		1320.1	812.7	1320.7	508.0	2.599				1875	1875	11
	B		1318.1	811.3	1318.4	507.1	2.599				1900	1976	10
	C		1318.7	811.4	1319.1	507.7	2.597				1740	1810	10
	Avg.						2.598	2.1	14.5	162.1		1887	10
5.82	5.5-A		1320.5	811.6	1320.6	509.0	2.594				1785	1785	13
	B		1326.1	814.9	1326.2	511.4	2.593				1685	1685	14
	C		1324.2	814.8	1324.4	509.6	2.599				1685	1685	13
	Avg.						2.595	1.4	15.0	161.9		1718	13
6.38	6.0-A		1327.6	813.7	1327.7	514.0	2.583				1490	1490	17
	B		1330.7	815.6	1330.9	515.3	2.582				1440	1440	17
	C		1329.3	814.4	1329.3	514.9	2.582				1580	1580	17
	Avg.						2.582	1.1	15.9	161.1		1503	17

*$Mg/m^3 = 0.01618\ lb/ft^3$
**N = 4.448 lbs.

†(See Conversion Table 4.3)

Inspector

Figure 4.9 Suggested Test Report Form Showing Test Data for a Typical Design by the Marshall Method.

Table 4.3 Stability Correlation Ratios

Volume of Specimen, cm³	Approximate Thickness of Specimen, mm	in.	Correlation Ratio
200 to 213	25.4	1	5.56
214 to 225	27.0	1 1/16	5.00
226 to 237	28.6	1 1/8	4.55
238 to 250	30.2	1 3/16	4.17
251 to 264	31.8	1 1/4	3.85
265 to 276	33.3	1 5/16	3.57
277 to 289	34.9	1 3/8	3.33
290 to 301	36.5	1 7/16	3.03
302 to 316	38.1	1 1/2	2.78
317 to 328	39.7	1 9/16	2.50
329 to 340	41.3	1 5/8	2.27
341 to 353	42.9	1 11/16	2.08
354 to 367	44.4	1 3/4	1.92
368 to 379	46.0	1 13/16	1.79
380 to 392	47.6	1 7/8	1.67
393 to 405	49.2	1 15/16	1.56
406 to 420	50.8	2	1.47
421 to 431	52.4	2 1/16	1.39
432 to 443	54.0	2 1/8	1.32
444 to 456	55.6	2 3/16	1.25
457 to 470	57.2	2 1/4	1.19
471 to 482	58.7	2 5/16	1.14
483 to 495	60.3	2 3/8	1.09
496 to 508	61.9	2 7/16	1.04
509 to 522	63.5	2 1/2	1.00
523 to 535	64.0	2 9/16	0.96
536 to 546	65.1	2 5/8	0.93
547 to 559	66.7	2 11/16	0.89
560 to 573	68.3	2 3/4	0.86
574 to 585	71.4	2 13/16	0.83
586 to 598	73.0	2 7/8	0.81
599 to 610	74.6	2 15/16	0.78
611 to 625	76.2	3	0.76

NOTES:

1. The measured stability of a specimen multiplied by the correlation ratio for the specimen thickness equals the corrected stability for a 63.5 mm (2 1/2-in.) specimen.
2. Volume-thickness relationship is based on a specimen diameter of 101.6 mm (4 in.).

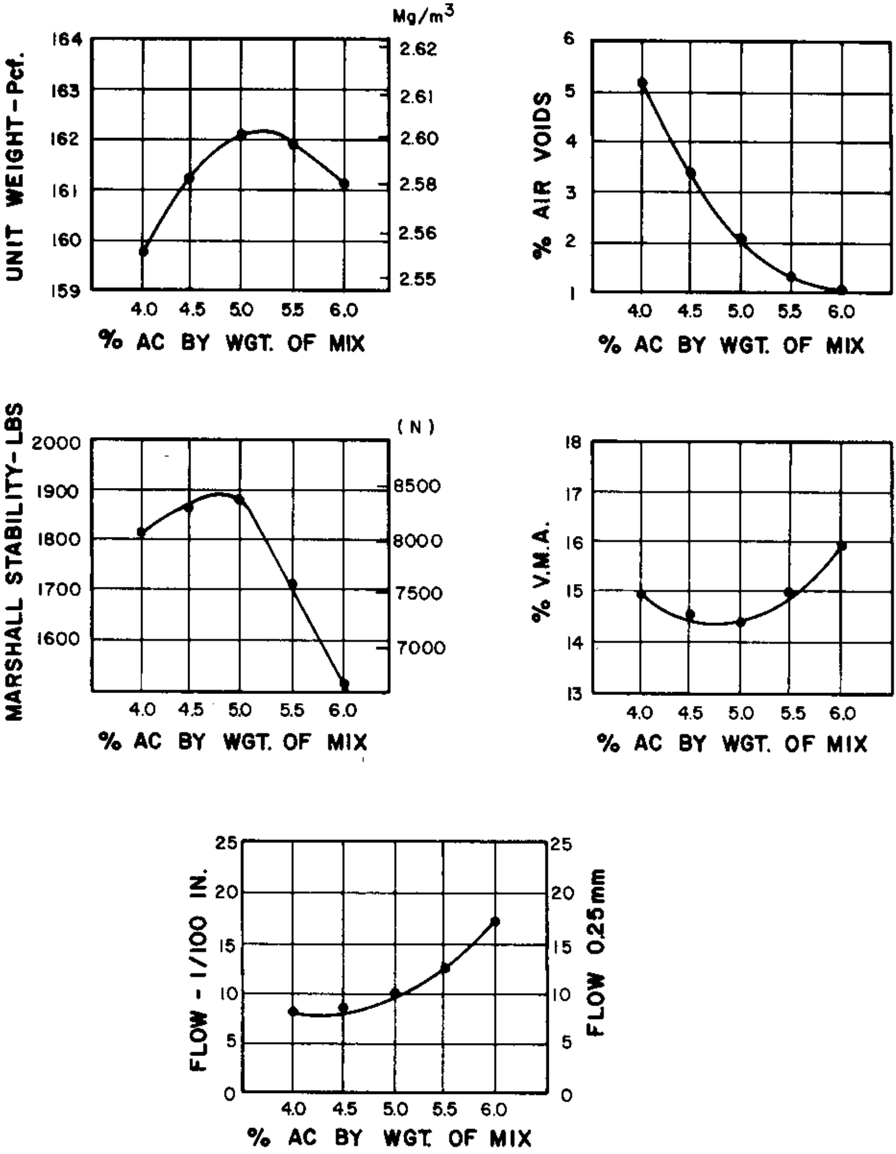


Figure 4.10 **Test Property Curves for Hot-Mix Design Data by the Marshall Method.**

- The curve for unit weight of total mix is similar to the stability curve, except that the maximum unit weight normally (but not always) occurs at a slightly higher asphalt content than the maximum stability.
- The percent of air voids decreases with increasing asphalt content, ultimately approaching a minimum void content.
- The percent voids in the mineral aggregate generally decrease to a minimum value then increase with increasing asphalt contents.

The optimum asphalt content of the asphalt paving mix is determined from data obtained as outlined above. Consideration is given to three of the test property curves illustrated in Figure 4.10 in making this determination. From these data curves, asphalt contents are determined which yield the following:

- Maximum stability
- Maximum unit weight
- Median of limits given in Table 4.4 for percent air voids

The optimum asphalt content of the mix is then the numerical average of the values for the asphalt content determined as noted above.

Table 4.4 Marshall Design Criteria

	Light Traffic[2]		**Medium Traffic[2]**		**Heavy Traffic[2]**	
	Surface & Base		**Surface & Base**		**Surface & Base**	
Marshall Method Mix Criteria[1]	**Min.**	**Max.**	**Min.**	**Max.**	**Min.**	**Max.**
Compaction, number of blows each end of specimen[3]	35		50		75	
Stability, Newton	3336	—	5338	—	8006	—
(lb.)	(750)	—	(1200)	—	(1800)	—
Flow, 0.25 mm (0.01 in.)[4]	8	18	8	16	8	14
Percent Air Voids[5]	3	5	3	5	3	5
Percent Voids in Mineral[6] Aggregate (VMA)	(Figure 4.11)					

NOTES:

[1] All criteria, not stability value alone, must be considered in designing an asphalt paving mix.
Hot-mix asphalt bases that do not meet these criteria when tested at 60°C (140°F) are satisfactory if they meet the criteria when tested at 38°C (100°F) and are placed 100 mm (4 in.) or more below the surface. This recommendation applies only to regions having a range of climatic conditions similar to those prevailing throughout most of the United States. A different lower test temperature may be considered in regions having more extreme climatic conditions.

[2] Traffic Classifications:
Light: Traffic conditions resulting a Design EAL $<10^4$.
Medium: Traffic conditions resulting a Design EAL between 10^4 and 10^6.
Heavy: Traffic conditions resulting in a Design EAL $>10^6$.

[3] Laboratory compactive efforts should closely approach the maximum density obtained in the pavement under traffic.

[4] The flow value refers to the point where the load begins to decrease.

[5] The portion of the asphalt cement lost by absorption into the aggregate particles must be allowed for when calculating percent air voids.

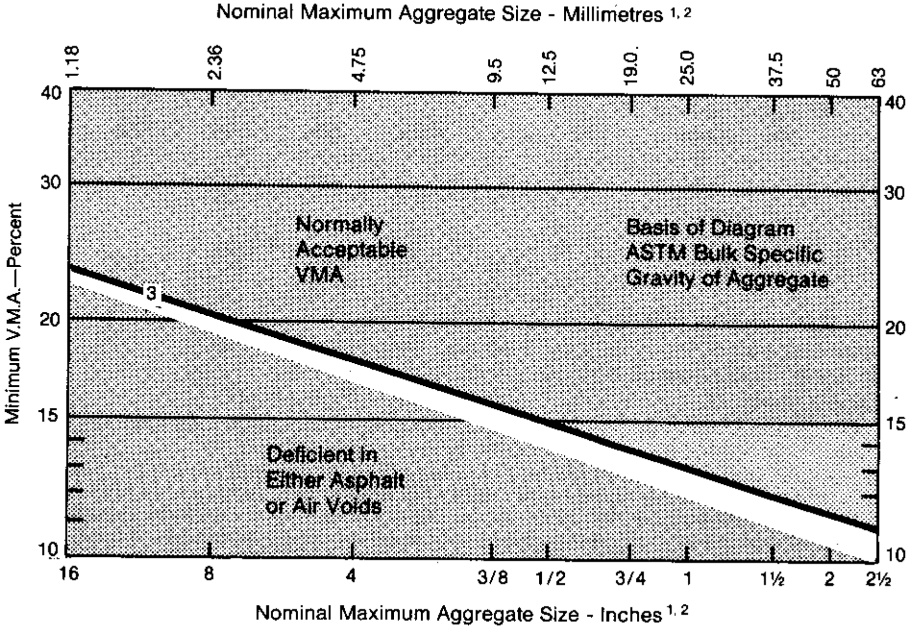
[6] Percent voids in the mineral aggregate is to be calculated on the basis of the ASTM bulk specific gravity for the aggregate.

Criteria for Satisfactory Paving Mix

Whether the asphalt paving mix will be satisfactory at the optimum asphalt content selected is determined by applying certain limiting criteria to test data for the mix at its optimum asphalt content. The design criteria in Table 4.4 are recommended by the Asphalt Institute.

In extreme cases when it is not possible or practicable, for economic or other reasons, to meet the requirements of the design criteria, a tolerance of one percent of air voids may be permitted. Under no circumstances, however, shall the allowable flow value be exceeded or shall the stability value be less than that required. It is to be emphasized that these variations should be allowed only under extreme conditions, unless service behavior with a specific aggregate mixture indicates such a paving mix to be satisfactory.

The selected mix design is usually the most economical one which will satisfactorily meet all of the established criteria. Mixes with abnormally high values of Marshall stability and abnormally low flow values are often less desirable because pavements of such mixes tend to be more rigid or brittle and may crack under heavy volumes of traffic. This is particularly true where base and subgrade deflections are such as to permit moderate to relatively high deflections of the pavement.



[1] Standard Specifications for *Wire Cloth Sieves for Testing Purposes*, ASTM Designation E11 (AASHTO Designation M92).

[2] For processed aggregate, the nominal maximum particle size is the largest sieve size listed in the applicable specification upon which any material is retained.

[3] Mixtures in the 1% tolerance band shall be permitted only when experience indicates that the mixture will perform satisfactorily and when all other criteria are met.

Figure 4.11 Minimum Percent Voids in the Mineral Aggregate.

Hveem Method of Mix Design

The Hveem method of mix design is a laboratory procedure based on two properties of the compacted specimen: cohesion and friction. Density and voids of the compacted specimen are established before measuring the stability with the stabilometer. The stabilometer is a triaxial testing device in which vertical loads are applied and resulting lateral pressures are measured.

Development and Application

The concepts of the Hveem method of designing paving mixtures have been advanced and developed under the direction of Francis N. Hveem, formerly Materials and Research Engineer for the California Department of Transportation. The test procedures and their application were developed through extensive research and correlation studies on asphalt highway pavements.

The Hveem method as developed and used by the California Department of Transportation is applicable to paving mixtures using both penetration and viscosity grades of asphalt and containing aggregates up to 25 mm (1 in.) maximum size. The method shown herein is applicable to the design of hot, dense-graded asphalt paving mixtures.

Hveem method test procedures have been standardized by the American Society for Testing and Materials. Test procedures are found in ASTM Designation D 1560, *Resistance to Deformation and Cohesion of Bituminous Mixtures by Means of Hveem Apparatus,* and ASTM Designation D 1561, *Preparation of Bituminous Mixture Test Specimens by Means of California Kneading Compactor.* Testing procedures presented herein are basically the same as those of the ASTM test methods. (It is recommended that where agencies refer to either AASHTO or ASTM, that the specific procedure be exactly followed.)

The procedure for the Hveem method as presented herein starts with the preparation of test specimens. Preliminary to this operation the following are required:

(1) That the materials proposed for use meet the requirements of the project specifications;

(2) That aggregate blend combinations are determined which meet the gradation requirements of the project specifications; and

(3) That an ample supply of aggregates is dried and sized into fractions.

These requirements are matters of routine testing, specifications, and laboratory technique, which must be considered but are not unique to any particular mix design method. It should be noted, however, that the maximum size aggregates used in the test mixes should not exceed 25 mm (1 in.). In the event that the specifications for the paving mix being considered require aggregate sizes greater than 25 mm (1 in.), oversize rock up to 25 percent may be screened out without marked effect on stabilometer values.

The Hveem method uses standard test specimens of 64 mm (2½ in.) height by 102 mm (4 in.) diameter; these are prepared using a specified procedure for heating, mixing, and compacting the asphalt-aggregate mixtures. The principal features of the Hveem method of mix design are the oil soak and Centrifuge Kerosene Equivalent (C.K.E.) tests on the aggregates to estimate the asphalt requirements of the mix, followed by a stabilometer test, a cohesiometer test*, a swell test, and a density-voids analysis on test specimens of the compacted paving mixtures. The stabilometer test utilizes a special triaxial-type testing cell for measuring the resistance of the compacted mix to lateral displacement under vertical loading, and the swell test measures the resistance of the mix to the action of water. The specimens are maintained at 60° C (140° F), for the stability tests, whereas the swell tests are performed at room temperature.

Determining the Approximate Asphalt Requirement

The first step in the Hveem method of mix design is to determine the approximate asphalt demand by the oil soak and Centrifuge Kerosene Equivalent tests,** and the calculated surface area. For a particular aggregate or blend of aggregates, the approximate asphalt content is determined by using a series of charts. These charts are presented herein, accompanied by typical examples to demonstrate their application.

(a) *Equipment*

The equipment (Figure 4.12) and materials required for determining the approximate asphalt requirement are as follows:

Sample Splitter, small, for obtaining representative samples of fine aggregate.

Pans, 114 mm (4½ in.) diameter × 25 mm (1 in.) deep.

Kerosene, 4 litres (1 gal).

Oil, SAE No. 10, lubricating, 4 litres (1 gal).

Beakers, 1500 ml.

Metal Funnels, 89 mm (3½ in.) top diameter, 114 mm (4½ in.) height, 13 mm (½ in.) orifice with piece of 2.00 mm (No. 10) sieve soldered to bottom of opening.

Timer.

Centrifuge, hand-operated, complete with cups, capable of producing 400 times gravity (a power-driven centrifuge is available from Soiltest, Inc., 2205 Lee Street, Evanston, Illinois 60602, Catalog No. AP-275 or equivalent.)

Filter Papers, 55 mm diam. (No. 611, Eaton-Dikeman Co., Mt. Holly Springs, Pennsylvania, or equivalent.)

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*The cohesiometer value is seldom used and the test, therefore, is not included in this manual. If the cohesiometer value is desired the test is described in ASTM *Standard Method of Test for Resistance to Deformation and Cohesion of Bituminous Mixtures by Means of Hveem Apparatus,* D 1560.

**The development of this method of determining optimum asphalt content is outlined in "Establishing the Oil Content for Dense-Graded Bituminous Mixtures" by F. N. Hveem, *California Highways and Public Works,* July-August, 1941. Note, where the C.K.E. equipment is not available, the approximate optimum asphalt content might be selected based on past experience. This should only occur under appropriate engineering judgement.

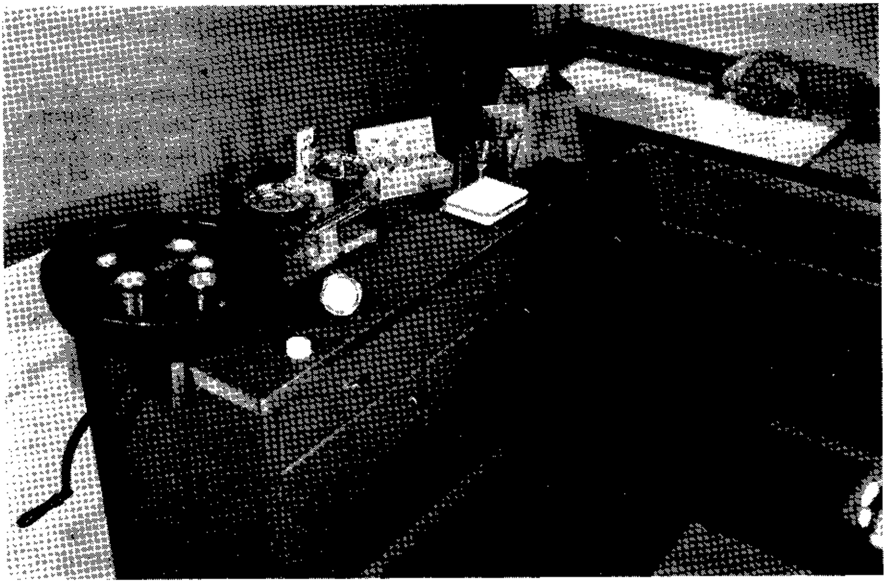


Figure 4.12 Apparatus for Hveem C.K.E. Tests.

(b) *Surface Area*

The gradation of the aggregate or blend of aggregates employed in the mix is used to calculate the surface area of the aggregates. This calculation consists of multiplying the total percent passing each sieve size by a "surface-area factor" as set forth in Table 4.5. Add the products thus obtained and the total will represent the equivalent surface area for the sample in terms of m^2/kg (ft^2/lb). It is important to note that all surface-area factors must be used in the calculation. Also, if a different series of sieves is used, different surface-area factors are necessary.

Table 4.5 Surface-Area Factors

Total Percent Passing Sieve No.	Maximum Size	4.75 mm (No. 4)	2.36 mm (No. 8)	1.18 mm (No. 16)	600 μm (No. 30)	300 μm (No. 50)	150 μm (No. 100)	75 μm (No. 200)
Surface-Area Factor,* m^2/kg (ft^2/lb.)	.41 (2)	.41 (2)	.82 (4)	1.64 (8)	2.87 (14)	6.14 (30)	12.29 (60)	32.77 (160)

*Surface area factors shown are applicable only when all the above-listed sieves are used in the sieve analysis.

The following tabulation demonstrates the calculation of surface area by this method.

Sieve Size	Percent Passing	×	S.A. Factor	=	Surface Area
19.0 mm (¾ in.)	100		.41 (2)		.41 (2)
9.5 mm (⅜ in.)	90				
4.75 mm (No. 4)	75		.41 (2)		.31 (1.5)
2.36 mm (No. 8)	60		.82 (4)		.49 (2.4)
1.18 mm (No. 16)	45		1.64 (8)		.74 (3.6)
600 μm (No. 30)	35		2.87 (14)		1.00 (4.9)
300 μm (No. 50)	25		6.14 (30)		1.54 (7.5)
150 μm (No. 100)	18		12.29 (60)		2.21 (10.8)
75 μm (No. 200)	10		32.77 (160)		3.28 (16.0)

Surface Area 9.98 m^2/kg
(48.7 ft^2/lb)

(c) *C.K.E. Test for Fine Aggregate*

(1) Place exactly 100 g of dry aggregate [representative of the passing 4.75 mm (No. 4) material being used] in the tared centrifuge cup assembly fitted with a screen and a disk of filter paper.

(2) Place bottom of centrifuge cup in kerosene until the aggregate becomes saturated.

(3) Centrifuge the saturated sample for two minutes at a force of 400 times gravity. (For the suggested centrifuge this force can be developed by turning the handle approximately 45 revolutions per minute.)

(4) Weigh sample after centrifuging and determine the amount of kerosene retained as a percent of the dry aggregate weight; this value is called the Centrifuge Kerosene Equivalent (C.K.E.). (Note: Duplicate samples are always prepared in order to balance the centrifuge and to check results. The average of the two C.K.E. values is used unless there is a large discrepancy, in which case the test is rerun.)

(5) If the specific gravity of samples is greater than 2.70 or less than 2.60 make a correction to the C.K.E. value using the formula at the bottom of the chart in Figure 4.13.

(d) *Surface Capacity Test (Oil Soak) for Coarse Aggregate*

(1) Place into a metal funnel, exactly 100 g of dry aggregate passing the 9.5 mm (⅜ in.) sieve and retained on the 4.75 mm (No. 4) sieve (this fraction is considered to be representative of the coarse aggregate in the mix).

(2) Immerse sample and funnel in a beaker containing SAE No. 10 lubricating oil at room temperature for 5 minutes.

(3) Drain for 2 minutes.

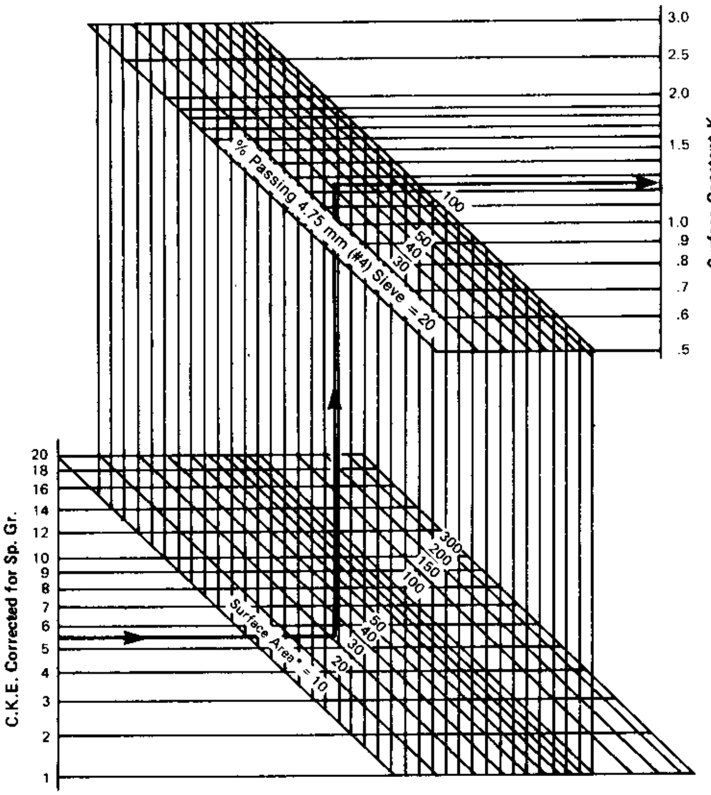
(4) Remove funnel and sample from oil and drain for 15 minutes at a temperature of 60° C (140° F).

(5) Weigh the sample after draining and determine the amount of oil retained as a percent of the dry aggregate weight. (Note: Duplicate samples

are prepared to check results. Average value is used unless there is a large discrepancy, in which case the test is rerun.)

(6) If the specific gravity is greater than 2.70 or less than 2.60 make a correction to the percent oil retained using the formula at the top of the chart in figure 4.14.

CHART FOR DETERMINING K_f FROM C.K.E.



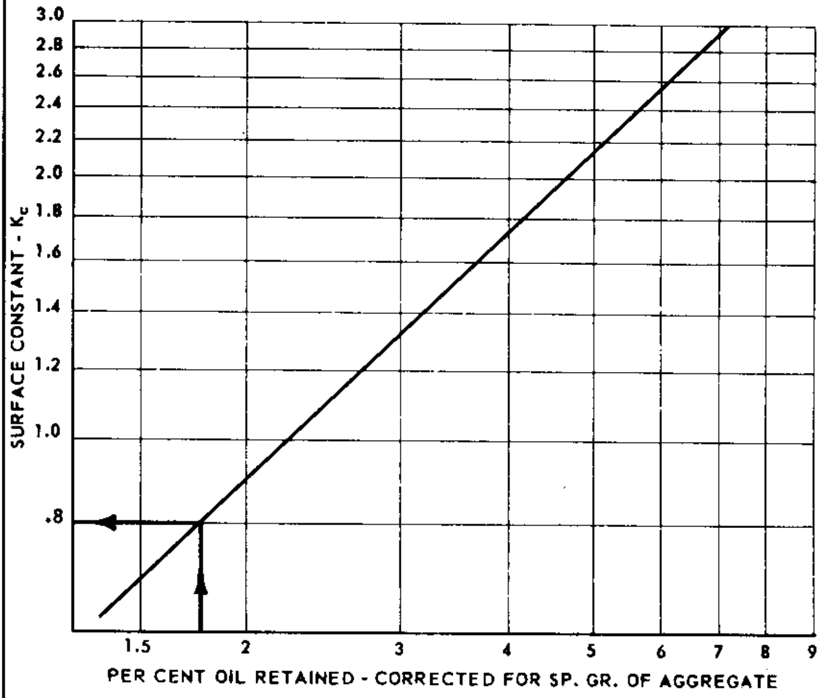
$$\text{C.K.E. Corrected} = \text{C.K.E.} \times \frac{\text{sp. gr. fine}}{2.65}$$

NOTE: Do not confuse this correction to C.K.E. with that used in Figure 4.16

*Surface area, $\frac{m^2}{kg} = 0.204816 \frac{ft^2}{lb}$

Courtesy of California Department of Transportation.

Figure 4.13 Chart for Determining Surface Constant for Fine Material, K_f, from C.K.E., Hveem Method of Design.



Material Used: Aggregate - Passing 9.5 mm (3/8"), Ret 4.75 mm (#4) Sieve

Oil - SAE 10

$$\% \text{ Oil Ret. Corrected} = \% \text{ Oil Ret.} \times \frac{\text{sp. gr. of Coarse Aggregate}}{2.65}$$

Courtesy of California Department of Transportation.

Figure 4.14 Chart for Determining Surface Constant for Coarse Material, K_c, from Coarse Aggregate Absorption, Hveem Method of Design.

(e) *Estimated Optimum Asphalt Content*

(1) Using the C.K.E. value obtained and the chart in Figure 4.13, determine the value K_f (surface constant for fine material).

(2) Using the percent oil retained and the chart in Figure 4.14, determine the value K_c (surface constant for coarse material.)

(3) Using the values obtained for K_f and K_c and chart in Figure 4.15, determine the value K_m (surface constant for fine-coarse aggregate combined). $K_m = K_f +$ correction to K_f. The correction to K_f obtained from Figure 4.15 is positive if $(K_c - K_f)$ is positive and is negative if $(K_c - K_f)$ is negative.

(4) The next step is to determine the approximate bitumen ratio for the mix based on cutback asphalts of RC-250, MC-250 and SC-250 grades. With values obtained for K_m, Surface Area and average specific gravity use CASE 2 procedures of Chart in Figure 4.16 to determine the oil ratio.

(5) Determine the approximate asphalt content (Figure 4.17) for mix, corrected for the grade to be employed, using the surface area of the sample, the grade of asphalt, and the oil ratio from Figure 4.16.

Example:

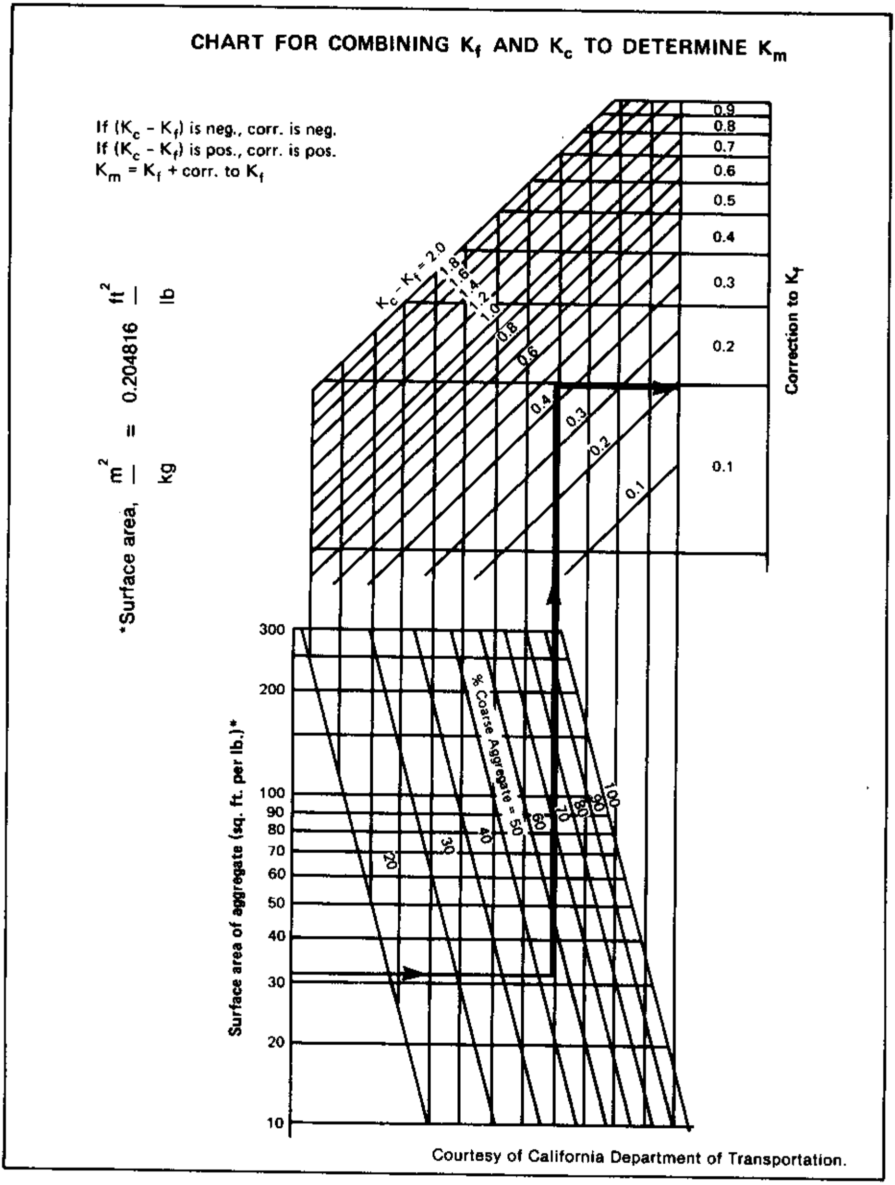
To demonstrate the use of the charts in Figures 4.13 through 4.17 assume the following conditions apply to a paving mix using AC-10 viscosity grade asphalt cement.

Specific Gravity, coarse = 2.45
Specific Gravity, fine = 2.64
Percent Passing 4.75mm No. 4 = 45

$$\text{Avg. Sp. Gr.} = \frac{100}{\frac{55}{2.45} + \frac{45}{2.64}} = 2.53$$

Surface Area of Aggregate Grading= 6.6 m^2/kg (32.4 ft^2/lb)
C.K.E. = 5.6
Percent Oil Retained, coarse = 1.9
(corrected for specific gravity, this value is 1.7 percent. *See* Figure 4.14)

From Figure 4.13 determine K_f as 1.25.
From Figure 4.14 determine K_c as 0.8.
From Figure 4.15 determine K_m as 1.15.
From Figure 4.16 determine the oil ratio for liquid asphalt as 5.2 percent.
From Figure 4.17 determine optimum asphalt content (bitumen ratio) for AC-10 asphalt as 6.1 percent by weight of dry aggregate.



Courtesy of California Department of Transportation.

Figure 4.15 Chart for Combining K_f and K_c to Determine Surface Constant for Combined Aggregate, K_m, Hveem Method of Design.

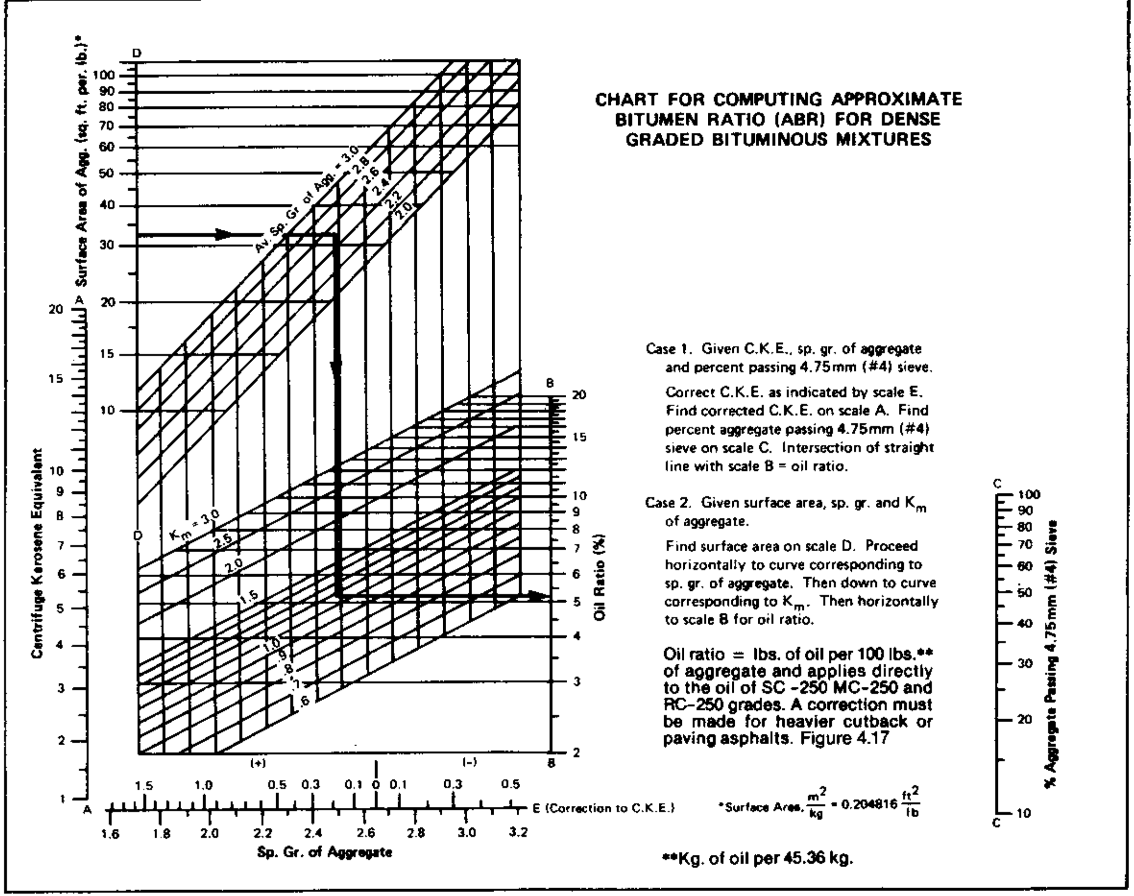


CHART FOR COMPUTING APPROXIMATE
BITUMEN RATIO (ABR) FOR DENSE
GRADED BITUMINOUS MIXTURES

Case 1. Given C.K.E., sp. gr. of aggregate and percent passing 4.75mm (#4) sieve.

Correct C.K.E. as indicated by scale E. Find corrected C.K.E. on scale A. Find percent aggregate passing 4.75mm (#4) sieve on scale C. Intersection of straight line with scale B = oil ratio.

Case 2. Given surface area, sp. gr. and K_m of aggregate.

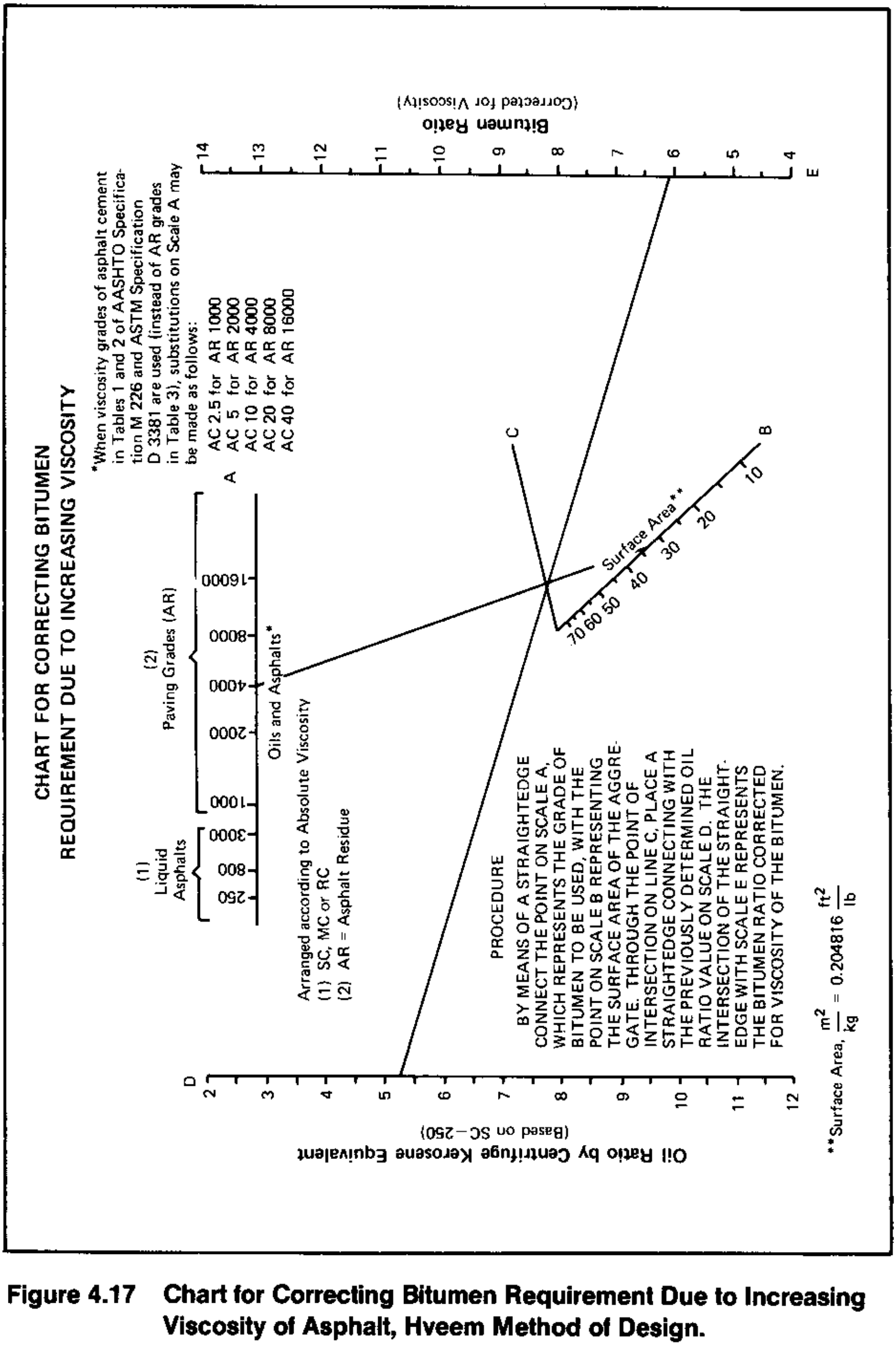
Find surface area on scale D. Proceed horizontally to curve corresponding to sp. gr. of aggregate. Then down to curve corresponding to K_m. Then horizontally to scale B for oil ratio.

Oil ratio = lbs. of oil per 100 lbs.** of aggregate and applies directly to the oil of SC -250 MC-250 and RC-250 grades. A correction must be made for heavier cutback or paving asphalts. Figure 4.17

*Surface Area, $\frac{m^2}{kg} = 0.204816 \frac{ft^2}{lb}$

**Kg. of oil per 45.36 kg.

Figure 4.16 Chart for Computing Oil Ratio for Dense-Graded Bituminous Mixtures, Hveem Method of Design.



*When viscosity grades of asphalt cement in Tables 1 and 2 of AASHTO Specification M 226 and ASTM Specification D 3381 are used (instead of AR grades in Table 3), substitutions on Scale A may be made as follows:

AC 2.5 for AR 1000
AC 5 for AR 2000
AC 10 for AR 4000
AC 20 for AR 8000
AC 40 for AR 16000

PROCEDURE

BY MEANS OF A STRAIGHTEDGE CONNECT THE POINT ON SCALE A, WHICH REPRESENTS THE GRADE OF BITUMEN TO BE USED, WITH THE POINT ON SCALE B REPRESENTING THE SURFACE AREA OF THE AGGREGATE. THROUGH THE POINT OF INTERSECTION ON LINE C, PLACE A STRAIGHTEDGE CONNECTING WITH THE PREVIOUSLY DETERMINED OIL RATIO VALUE ON SCALE D. THE INTERSECTION OF THE STRAIGHTEDGE WITH SCALE E REPRESENTS THE BITUMEN RATIO CORRECTED FOR VISCOSITY OF THE BITUMEN.

**Surface Area, $\frac{m^2}{kg} = 0.204816 \frac{ft^2}{lb}$

Figure 4.17 Chart for Correcting Bitumen Requirement Due to Increasing Viscosity of Asphalt, Hveem Method of Design.

Preparation of Test Specimens for Hveem Procedure

In designing a paving mix by the Hveem method a series of stabilometer test specimens is prepared with a range of asphalt contents both above and below the approximate asphalt content (AAC) as determined in the previously described procedure.

For hot-mix designs using an average aggregate, prepare one specimen with the AAC, two above the AAC amount in 0.5 percent increments, and one 0.5 percent below the AAC amount. (Total of four specimens, each with a different asphalt content). For mixes thought to be critical, (i.e., very sensitive to slight changes in asphalt content) the increments in asphalt content are lowered to 0.3 percent and tests are scheduled for the approximate asphalt content; three above the AAC amount in 0.3 percent increments, and one 0.3 percent below the AAC amount. For highly absorptive aggregates, increase the increments in asphalt content to 1.0 percent and use more specimens as necessary. Regardless of these general rules for preparing stabilometer specimens, the series of test specimens should have at least one specimen containing an excess of asphalt as indicated by moderate or heavy flushing after compaction.

In addition swell test specimens are prepared in duplicate *at the optimum asphalt content as determined from tests on the series of specimens prepared for stabilometer tests.* Thus for a normal mix design study a total of six test specimens will usually be required. Although each test specimen will normally require only 1.2 kg (2¼ lb) of aggregate, the minimum aggregate requirements for a series of test specimens should be at least 18 kilograms (40 lb.) to provide for additional tests that may be required.

In lieu of a swell test some agencies allow the substitution of an alternative procedure such as immersion/compression or other moisture susceptibility tests mentioned in Chapter 3.

The equipment required for the preparation of test specimens is as follows:

Pans, 250 millimetres (10 in.) diameter × 50 millimetres (2 in.) deep, for quartering and mixing fine aggregate.

Pans, 200 mm (8 in.) diameter × 45 mm (1¾ in.) deep, for batching and heating aggregates.

Pans, 305 mm (12 in.) diameter × 64 mm (2½ in.) deep for mixing aggregate and asphalt.

Pans, 280 mm (11 in.) × 180 mm (7 in.) × 40 mm (1½ in.) for curing mix.

Sample Splitter, large, for mixing and quartering fine aggregate.

Hot Plate, electric, at least 460 mm (18 in.) × 305 mm (12 in.), plate surface, for heating aggregates, asphalt and equipment as required.

Oven, large, thermostatically controlled, capable of 110° C (230° F) temperature.

Oven, large, thermostatically controlled, capable of 60° C (140 °F) temperature.

Oven, large, for drying and preheating, capable of temperatures up to 163° C (325° F).

Scoop, large, for handling hot aggregates.

Beakers, 800 ml, for adding asphalt.

Thermometer, armored, 38° C (100° F) to 204° C (400° F).

Balance, 20-kg capacity, sensitive to 1.0 g for weighing aggregates and asphalt.

Mixing Trowel, small pointed.

Mixing Spoon, large.

Mechanical Mixer (optional).

Mechanical Compactor designed to consolidate the material by a series of individual "kneading action" impressions made by a roving ram having a face shaped as a sector of a 101.6 mm (4 in.) diameter circle. The compactor must be capable of exerting a force of 34.5 kPa (500 psi) under the tamper foot. Accessories with the compactor should include two mold holders, a feeder trough 460 mm (18 in.) long × 102 mm (4 in.) wide × 64 mm (2½ in.) deep and a paddle shaped to fit trough, and a bullet-nosed steel rod 9.5 mm (⅜ in.) diameter × 405 mm (16 in.) long.

Compaction Molds, steel, 101.6 mm (4 in.) inside diameter × 127 mm (5 in.) high × 6.4 mm (¼ in.) wall thickness.

Paper Disks, heavy paper, 100 mm ($3^{15}/_{16}$ in.) diameter, to place in bottom of mold during compaction.

Compression Machine, hydraulic 222.4 kN (50,000 lb) capacity.

Gloves, heavy and sturdy, for handling hot equipment.

Specimen preparation is outlined in the following steps.

(a) *Determining batch weights*

Compute batch weights for the blend and gradation of aggregates desired.

The dry weight of the aggregate for the stabilometer specimens is that which will produce a compacted specimen of 63.5 ± 1.3 mm (2.5 ± 0.05 in.) in height. This will normally be about 1200 grams. To determine this exact batch weight it is generally desirable to prepare a trial specimen prior to preparing the aggregate batches. If the trial specimen height falls outside the limits, the amount of aggregate used for the specimen may be adjusted as follows:

For International System of Units (SI),

$$\text{Adjusted mass of aggregate} = \frac{63.5\ (\text{mass of aggregate used})}{\text{Specimen height (mm) obtained}}$$

- or -

For U.S. Customary Units,

$$\text{Adjusted weight of aggregate} = \frac{2.5\ (\text{weight of aggregate used})}{\text{Specimen height (in.) obtained}}$$

The dry weight for the aggregate for the swell specimen is 1000 grams.

(b) *Batch Mix Preparation*

(1) Weigh the various-sized fractions of dry aggregates into suitable pans in accordance with accumulative batch weights.

(2) Thoroughly mix each individual batch of aggregate and place in oven to preheat to desired mixing temperature. Asphalt should be preheated at the same time. The temperature of the aggregate and asphalt at the time mixing begins is indicated below for the paving grade of asphalt cement being used.

Grade	Temperature Range Minimum	Maximum
AC-2.5, AR-1000, or 200-300 Pen.	107° C (225° F)	121° C (250° F)
AC-5, AR-2000, or 120-150 Pen.	121° C (250° F)	135° C (275° F)
AC-10, AR-4000, or 85-100 Pen.	135° C (275° F)	149° C (300° F)
AC-20, AR-8000, or 60-70 Pen.	149° C (300° F)	163° C (325° F)
AC-40, AR-16000, or 40-50 Pen.	149° C (300° F)	163° C (325° F)

(3) When the aggregates and asphalt have reached the mixing temperature indicated above, form a crater in the aggregates and weigh in asphalt in accordance with accumulative batch weights.

(4) Place pan containing aggregates and asphalt for batch mix on hot plate to maintain mixing temperature. Vigorously hand-mix aggregates and asphalt with pointed trowel until all particles are coated. Take special precaution not to overheat the materials. The mixing operation may also be accomplished by mechanical mixing.

(5) After mixing is complete, transfer the batch mix to a suitable flat pan and place in oven for a 15-hour curing period at 60° C ± 2.8° C (140° F ± 5° F). The oven used for curing should preferably be equipped for forced draft air circulation.

(6) After curing is complete, place batch mix in heating oven and reheat mixture to 110° C (230° F). The batch mix is then ready for compaction.

(7) The compaction of the test specimen is accomplished by means of the mechanical compactor that imparts a kneading action consolidation by a series of individual impressions made with a ram having a face shaped as a sector of a 101.6 mm (4 in.) diameter circle. At each application of the ram a pressure of 3.45 MPa (500 psi) is applied, subjecting the specimen to a kneading action without impact over an area of approximately 2000 square millimetres (3.1 in.^{2}). Each pressure is maintained for approximately 2/5ths of a second. The details of the compaction procedure are described in the next paragraphs.

(c) *Procedure for Stabilometer Specimens*

(1) Preheat compaction molds to approximately the temperature to be used for the compaction of the mix.

(2) Heat compactor foot to a temperature that will prevent the mix from adhering to it. Temperature of the compactor foot may be controlled by a variable transformer.

(3) Place the compaction mold in position in the mold holder with a 101.6 mm (4 in.) diameter paper disc inserted to cover the base plate. In order to

have the base plate act as a free-fitting plunger during the compaction operation, a steel shim 6.4 mm (¼ in.) thick × 19.0 mm (¾ in.) wide × 63.5 mm (2½ in.) long is temporarily placed under the edge of the mold.

(4) Spread the prepared mixture (see batch mix preparation) uniformly in feeder trough. With a paddle made to fit the shape of the trough, transfer approximately one-half of the mixture to the compaction mold (*See* Figure 4.18).

(5) Rod the portion of the mix in the mold 20 times in the center of the mass and 20 times around the edge with the bullet-nosed steel rod. Transfer the remainder of the sample to the mold and repeat the rodding procedure (See Figure 4.19).

(6) Place mold assembly into position on the mechanical compactor (Figure 4.20) and apply approximately 20 tamping blows at 1.7 MPa (250 psi) pressure to accomplish a semi-compacted condition of the mix so that it will not be unduly disturbed when the full load is applied. The exact number of blows to accomplish the semi-compaction shall be determined by observation. The number of blows may vary between 10 and 50, depending upon the type of material, and in some instances it may not be possible to accomplish the compaction in the mechanical compactor because of undue movement of the mixture under the compactor foot. In these instances use a 177.93 kilonewton (40,000 lb.) static load applied by the double plunger method, in which a free-fitting plunger is placed below the sample as well as on top. Apply the load at the rate of 1.3 mm (0.05 in.) per minute and hold for 30 ± 5 seconds.

(7) After the semi-compaction, remove shim and release mold tightening screw sufficiently to allow free up-and-down movement of mold.

(8) To complete compaction in the mechanical compactor, increase compactor foot pressure to 3.45 MPa (500 psi) and apply 150 tamping blows.

(9) The mold and specimen are placed in the 60° C (140° F) oven for 1½ hours, after which a "leveling-off" load of 6.9 MPa (1,000 psi) is applied by the "double-plunger" method [head speed 0.105 mm per second (0.25 in/min)] and released immediately. (Note: The specimen shall not be pushed to the opposite end of the mold.)

(d) *Procedure for Swell Test Specimens*

(1) Prepare compaction mold by placing a paraffin-impregnated strip of ordinary wrapping paper 19 mm (¾ in.) wide, around the inside of mold 13 mm (½ in.) to 19 mm (¾ in.) from the bottom to prevent water from escaping from between the specimen and the mold during the water immersion period of the test. The paper strip is dipped in melted paraffin and applied while hot. Compaction molds are not preheated for swell test specimens.

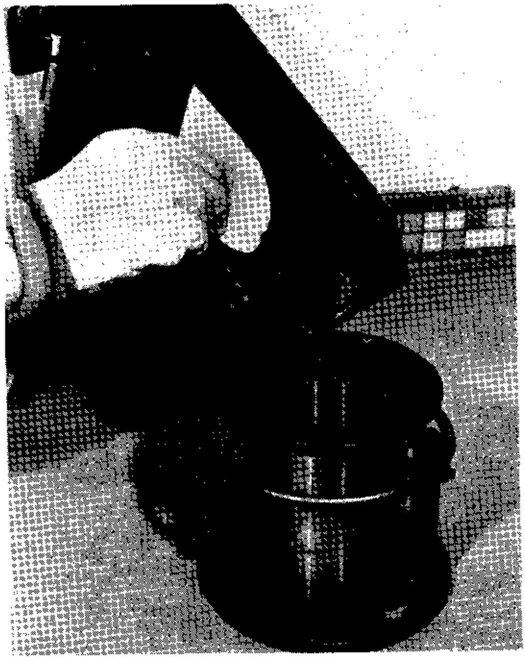


Figure 4.18 Transfer of Mix to Mold.

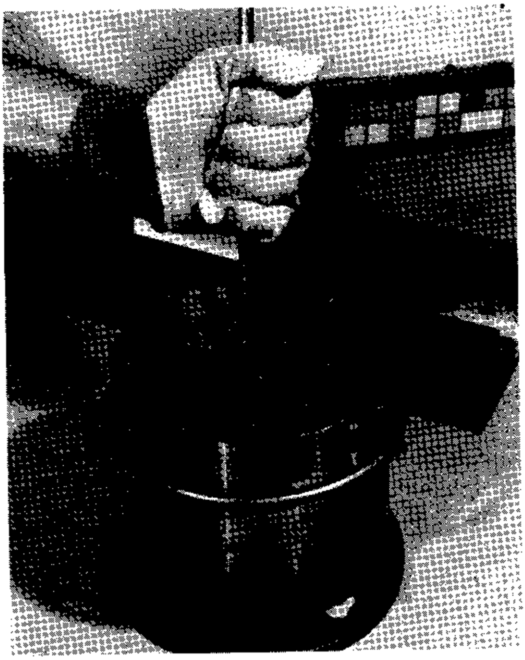


Figure 4.19 Rodding Mix in Mold.

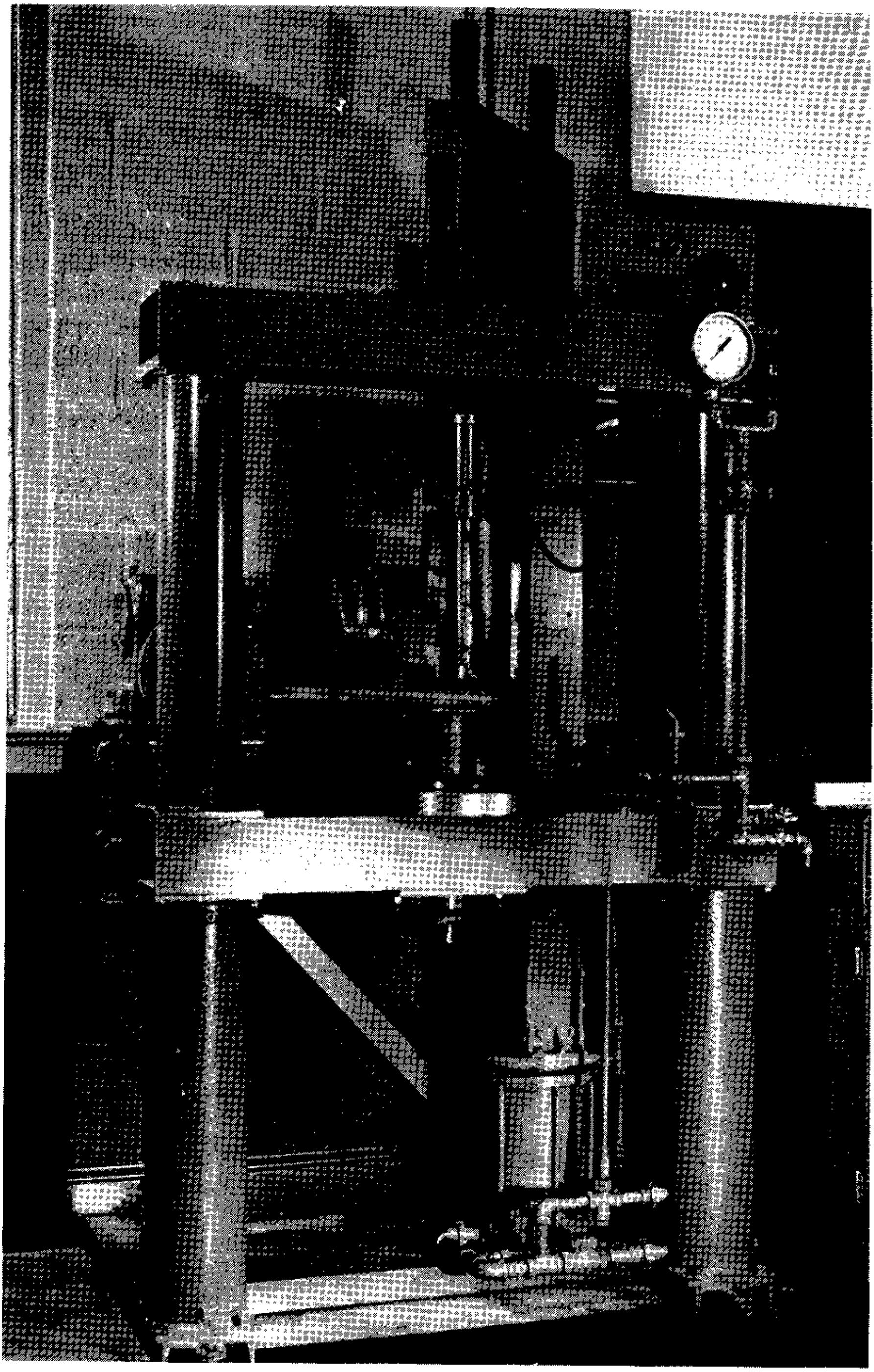
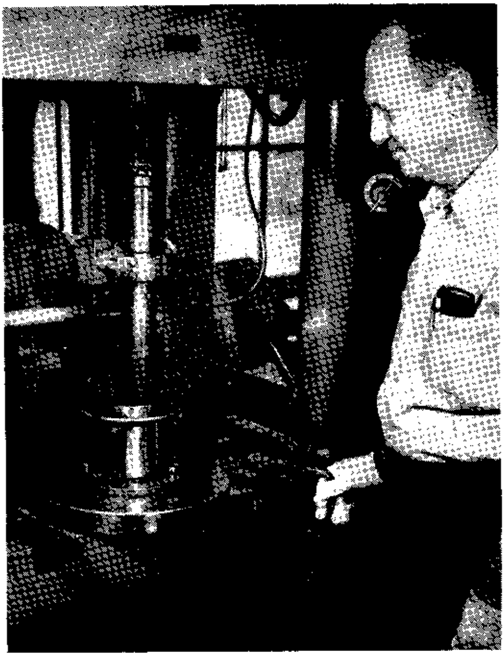
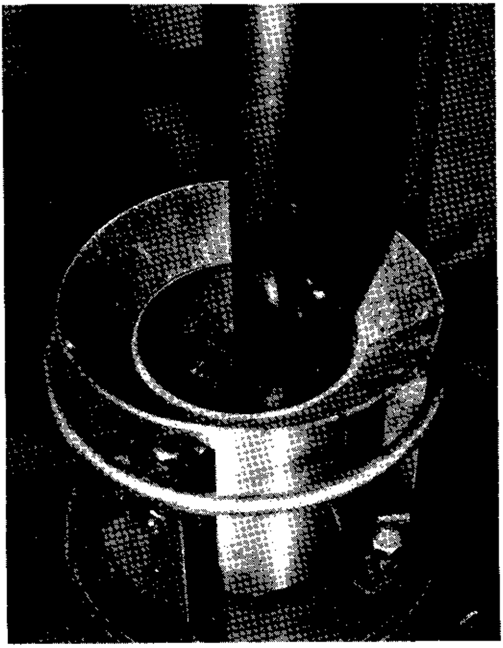


Figure 4.20 Mechanical Kneading Compactor for the Preparation of Test Specimens.



(b)



(c)

(2) The remainder of the compaction procedure for swell test specimens is the same as for the stabilometer test specimens with the following exception:

That when compaction is completed in the mechanical compactor, remove mold and specimen from compactor, invert mold and push specimen to the opposite end of mold. Apply a 6.9 MPa (1,000 psi) static load [head speed 6.3 mm per minute (0.25 in./min)] with the "original" top surface supported on the lower platen of testing press. It is advisable to place a piece of heavy paper under the specimen to prevent damage to this lower platen.

Hveem Test Procedure

In the Hveem method the compacted test specimens are used in the following tests and analyses and are normally performed in the order listed:

- Stabilometer Test
- Bulk Density Determination
- Swell Test

The equipment required for the testing of the 102 mm (4 in.) diameter specimens is as follows:

Bronze Disks, perforated, 98.4 mm ($3^7/_8$ in.) diameter × 3.2 mm ($^1/_8$ in.) thick, with adjustable stem, for swell measurement. *(See* Figure 4.21).

Dial Gauge, mounted on tripod, with reading accuracy to 0.025 mm (0.001 in.) *(See* Figure 4.22).

Scale, graduated to read the volumetric contents of a 101.6 mm (4 in.) inside-diameter mold at 25 ml intervals, for measuring percolation of water during swell test.

Pans, aluminum, 190 mm (7½ in.) diameter × 65 mm (2½ in.) deep.

Hveem Stabilometer (see Figures 4.21 and 4.22), complete with accessories including adjustable base, assembly tool, steel follower, and rubber bulb for introducing air into system.

Scale, or other measuring device to accurately determine height of compacted test specimen.

(a) *Stabilometer Test*

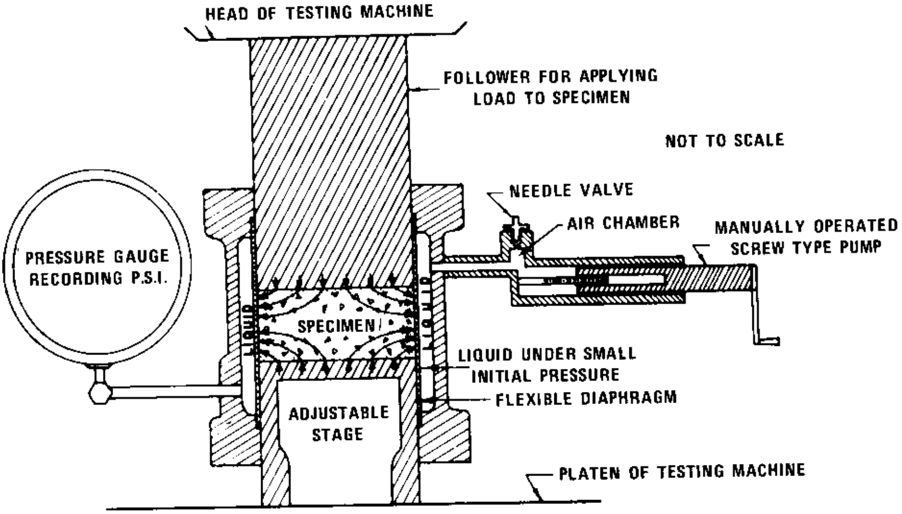
(1) Place specimens for stabilometer tests (compacted and contained in mold) in oven at 60°C (140°F) for a minimum period of one hour before testing.

(2) Adjust compression machine for a head speed of 0.02 mm per second (0.05 in/min) with no load applied.

(3) Check displacement of stabilometer with metal dummy specimen and if necessary adjust to give 2.00 ± 0.05 turns.

(4) Adjust the stabilometer base so that the distance from the bottom of the upper tapered ring to the top of the base is 89 mm (3.5 in.).

(5) Every effort should be made to fabricate test specimens with an overall height between 61 mm (2.40 in.) and 66 mm (2.60 in.); however, if the height is outside of this range the stabilometer value should be corrected as indicated in Figure 4.23.


HEAD OF TESTING MACHINE
FOLLOWER FOR APPLYING LOAD TO SPECIMEN
NOT TO SCALE
NEEDLE VALVE
AIR CHAMBER
MANUALLY OPERATED SCREW TYPE PUMP
PRESSURE GAUGE RECORDING P.S.I.
SPECIMEN
LIQUID
LIQUID
LIQUID UNDER SMALL INITIAL PRESSURE
FLEXIBLE DIAPHRAGM
ADJUSTABLE STAGE
PLATEN OF TESTING MACHINE

NOTE– The specimen is given lateral support by the flexible sidewall, which transmits horizontal pressure to the liquid. The magnitude of the pressure can be read on the gage.

Figure 4.21 Diagram Showing Principal Features of Hveem Stabilometer.

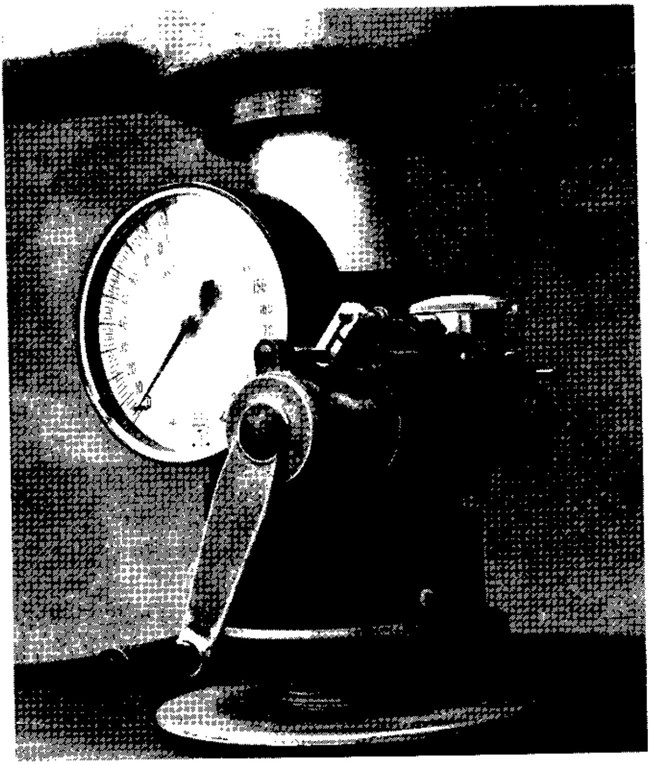


Figure 4.22 Hveem Stabilometer.

CHART FOR CORRECTING STABILOMETER VALUES TO SPECIMEN HEIGHT OF 64 mm (2.50 in.)

Height correction should be made using the table and chart below.

Example: Overall height of 69 mm (2.74 in.) select correction curve "B". Stabilometer value uncorrected = 35 Stabilometer value corrected = 38.

Overall Specimen Ht.	Correction Curve
71mm to 76mm (2.80 in. to 3.00 in.)	A
66mm to 70mm (2.60 in. to 2.79 in.)	B
61mm to 65mm (2.40 in. to 2.59 in.)	C
56mm to 60mm (2.20 in. to 2.39 in.)	D
51mm to 55mm (2.00 in. to 2.19 in.)	E

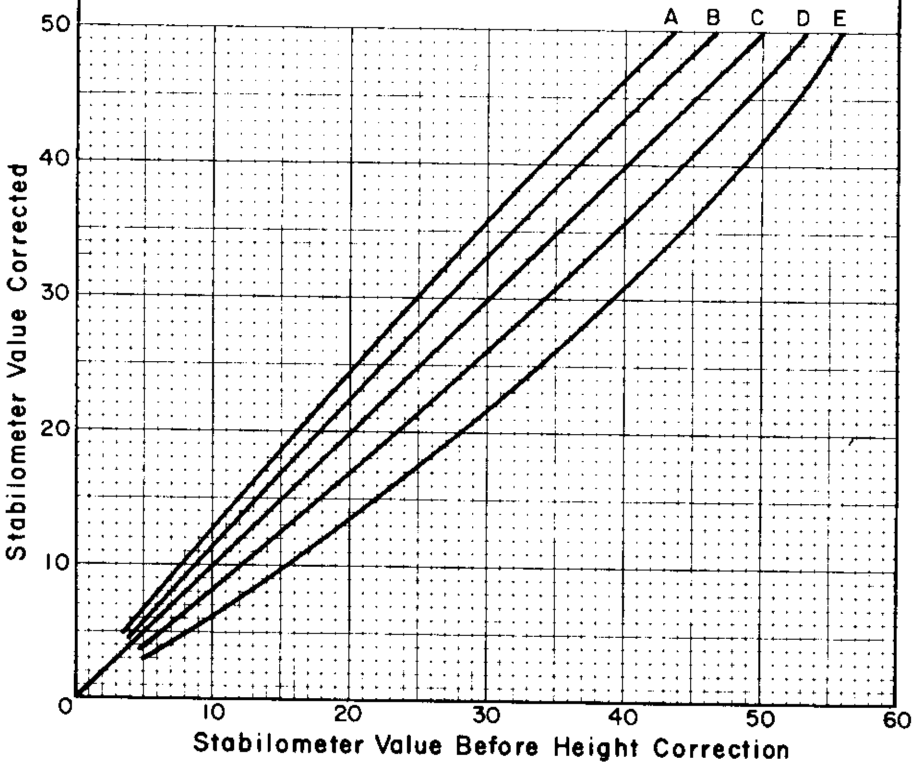


Figure 4.23 **Chart for Correcting Stabilometer Values to Effective Specimen Height of 64 mm (2.5 in.).**

(6) Remove mold containing specimen from oven and place on top of stabilometer. Using the plunger, hand lever, and fulcrum, force the specimen from the mold into the stabilometer. Take care that the specimen goes in straight and is firmly seated.

(7) Place follower on top of specimen and position the entire assembly in compression machine for testing.

(8) Using a displacement pump, raise the pressure in the stabilometer system until test gauge reads exactly 34.5 kPa (5 psi). Then tap test gauge lightly to assure an accurate reading.

(9) Close displacement pump valve, taking care not to disturb the 34.5 kPa (5 psi) initial pressure. (This step is omitted on stabilometers that are not provided with the displacement pump valve.)

(10) Apply test loads with compression machine using a head speed of 0.02 mm per second (0.05 in/min). Record readings of test gauge at test loads of 2.22 kN (500 lb), 4.45 kN (1,000 lb) and each 4.45 kN (1,000 lb) thereafter up to a maximum of 26.69 kN (6,000 lb).

(11) Immediately after recording the reading under maximum load [26.69 kN (6,000 lb)], reduce total load on specimen to 4.45 kN (1,000 lb).

(12) Open the displacement pump angle valve and by means of the displacement pump, adjust test gauge to 34.5 kPa (5 psi) (this will result in a reduction in the applied press load which is normal and for which no compensation is made).

(13) Adjust dial gauge on pump to zero by means of small thumbscrew.

(14) Turn displacement pump handle rapidly (two turns per second) and smoothly to right (clockwise) until a pressure of 690 kPa (100 psi) is recorded on test gauge. (During this operation the load registered on the testing press will increase and in some cases exceed the initial 4.45 kN (1,000 lb) load. This change in load is normal and no adjustment or compensation is required under the present procedure). Record the exact number of turns required to increase the test gauge reading from 34.5 kPa (5 psi) to 690 kPa (100 psi) as *displacement* on specimen (2.5 mm (0.1 in.) dial reading is equivalent to one turn displacement).

(15) After recording the displacement, first remove the test load and reduce pressure on test gauge to zero by means of displacement pump; then back off displacement pump an additional three turns and remove specimen from stabilometer chamber.

(b) *Bulk Density Determination*

The bulk density test is performed on the specimens after the completion of the stabilometer tests and as soon as the specimens have cooled to room temperature. The procedure for this test is presented in ASTM Designation D 1188, *Bulk Specific Gravity of Compacted Bituminous Mixtures Using Paraffin-Coated Specimens* or ASTM Designation D 2726, *Bulk Specific Gravity of Compacted Bituminous Mixtures Using Saturated Surface-Dry Specimens.*

(c) *Swell Test*

(1) Allow compacted specimen for swell test to stand at room temperature for at lest one hour (this is done to permit rebound after compaction).

(2) Place mold and specimen in 190 mm (7½ in.) diameter × 64 mm (2½ in.) deep aluminum pan. *(See* Figure 4.24).

(3) Place perforated bronze disk on specimen, position tripod with dial gauge on mold, and set adjustable stem to give a reading of 2.54 mm (0.100 in.) on dial gauge. *(See* Figure 4.24).

(4) Introduce 500 ml of water to mold on top of specimen and measure distance from top of mold to surface water with graduated scale.

(5) After 24 hours again read dial gauge and record the change as *swell* to nearest 0.025 mm (0.001 in.). Also, again measure distance from top of mold to surface of water with graduated scale and record the change as *permeability* or the amount of water in millimeters that percolates into and/or through the test specimen.

Interpretation of Hveem Procedure Test Data

There are no calculations required for the swell test since the results are reported directly as differences. The remainder of the calculations are as follows:

(a) *Stabilometer Value,* calculate as:

$$S = \frac{22.2}{\dfrac{P_h D_2}{P_v - P_h} + 0.222}$$

where S = stabilometer value
D_2 = displacement on specimen
P_v = vertical pressure [typically 400 psi = 5000 lb total load]
P_h = horizontal pressure = stabilometer pressure gauge reading taken at the instant P_v is 400 psi (5000 lb) total load

(Formula is an empirical analysis using U.S. customary units.)

(b) *Density and Voids Analysis*

Using the bulk specific gravity of the test specimens, and maximum specific gravity of the paving mixture, compute the percent air voids as illustrated in Figure 4.25 and more fully described in Section 4.2. Figure 4.25 is also useful as a suggested test report form showing test data for a typical mix design by the Hveem method. Data can then be plotted as shown in Figure 4.26.

Criteria for Satisfactory Paving Mix

The suitability of the hot-mix design by the Hveem method is determined on the basis of whether the asphalt content and aggregate grading will satisfy the following requirements of Table 4.6.

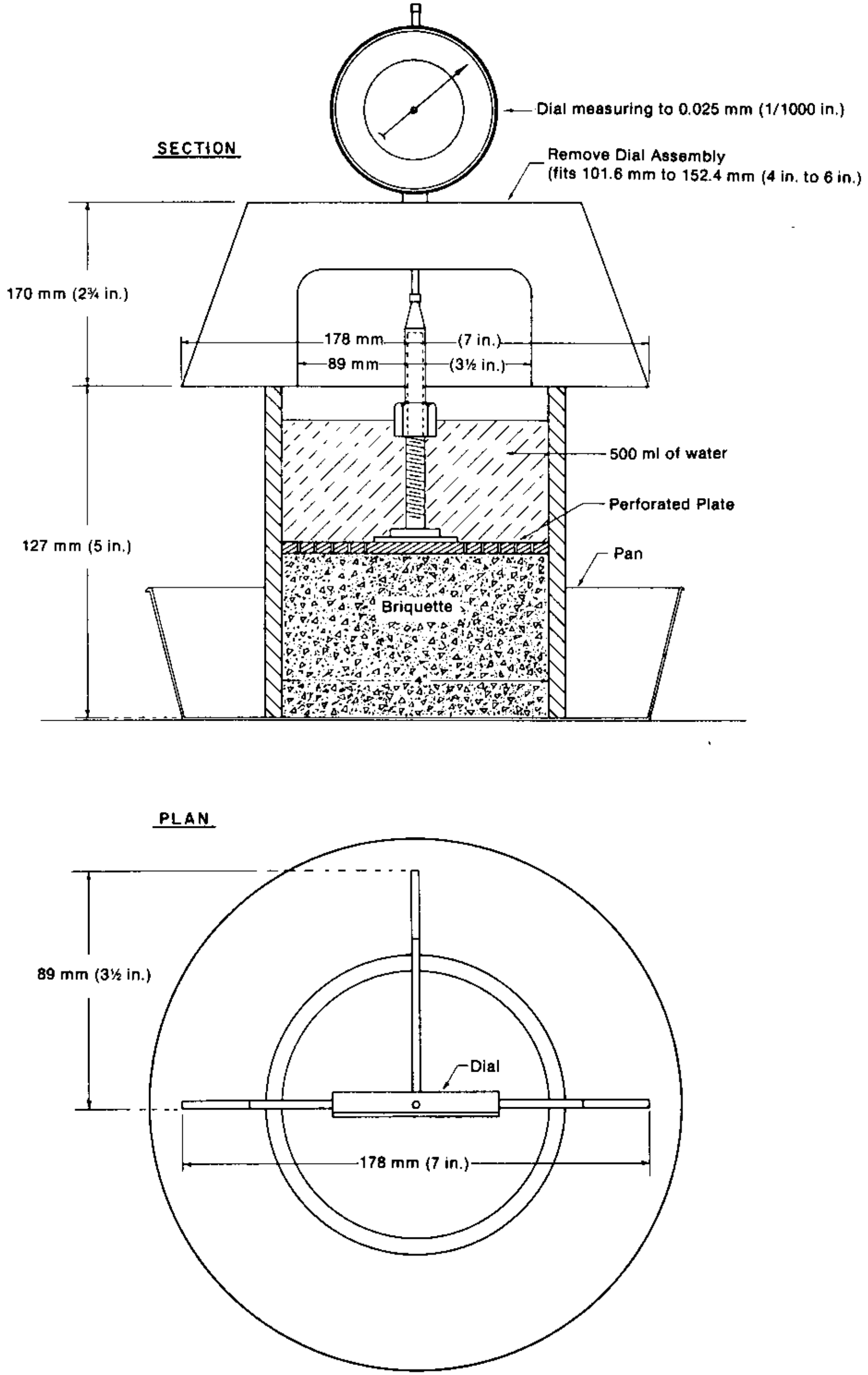


Figure 4.24 Swell Test Apparatus.

Trial Mix Series: 1-B
53% CA; 47% FA

HOT-MIX DESIGN DATA
by the
HVEEM METHOD

Project: F1-008-8(3)
Location: Rye - South
Date: August 4, 1982

Sp. Gr. Asp. Cem. 1.012 Asp. Cem. AC-10
Avg. Bulk Sp. Gr. Agg. = 2.760

Lab. No. for Asp. Cem. Used: 53-0741
Lab. Nos. for Agg. Used: 53-1252; 53-1253

GRADATION, CKE, and PERCENT ASPHALT

Sieve Size	37.5	75.0	19.0	12.5	9.5	4.75	2.36	1.18	600	300	150	75
	1½	1	¾	½	3/8	4	8	16	30	50	100	200
Specification Limits			100	100 / 80	90 / 70	70 / 50	50 / 35	/	29 / 18	/	16 / 8	10 / 4
% Passing			100	91	76	60	42	32	23	16	12	6
S. A. Factors						.41	.82	1.64	2.87	6.14	12.29	32.77
Surface Area, m^2/kg*					.41	.25	.34	.52	.66	.98	1.47	1.97

CKE: FA = 2.8; CA = 2.8 K_f = 1.0; K_c = 1.3; K_m = 1.0; Total SA 6.6 m^2/kg (32.3 ft^2/lb)
Estimated % Asp. Cem. by Wgt. of Agg. using CKE Tests only 5.5
Recommended % Asp. Cem. by Wgt. of Agg. using Mix Design Criteria 5.0

Specimen Identification	A	B	C	D
% Asp. Cem. by Wgt. of Agg.	5.0	5.5	6.0	6.5
% Asp. Cem. by Wgt. of Mix	4.76	5.21	5.66	6.10
Wgt. in Air-grams	1211.0	1223.3	1230.8	1235.9
Wgt. in Water-grams	714.9	723.8	727.6	733.3
Bulk Volume-cc.	496.1	499.5	503.2	502.6
Bulk Sp. Gr.	2.441	2.449	2.446	2.459
Max Sp. Gr.	2.559	2.540	2.522	2.504
% Voids-Total Mix	4.6	3.6	3.0	1.8
Unit Wgt. - Mg/m^3 (lb/ft^3)	2.439 (152.3)	2.448 (152.8)	2.446 (152.6)	2.457 (153.4)

Total Load kN	(lbs)	Unit Load-MPa	(psi)	STABILOMETER			
2.22	(500)	0.28	(40)	9	9	9	10
4.45	(1000)	0.55	(80)	12	12	15	16
8.90	(2000)	1.10	(160)	15	16	24	26
13.34	(3000)	1.65	(240)	21	22	30	38
17.79	(4000)	2.21	(320)	28	30	42	55
22.24	(5000)	2.76	(400)	36	39	55	83
26.69	(6000)	3.31	(480)	50	52	62	105
Displacement-turns				2.40	2.50	2.46	2.50
Stability Value				48	45	36	25

COHESIOMETER

Temperature - °C (°F) Effective Height - mm (in.) Shot Weight - grams Cohesiometer Value					

Jones
Inspector

*Surface Area ft^2/lb = m^2/kg ÷ 0.204816

Figure 4.25 Suggested Test Report Form Showing Test Data for a Typical Mix Design by the Hveem Method.

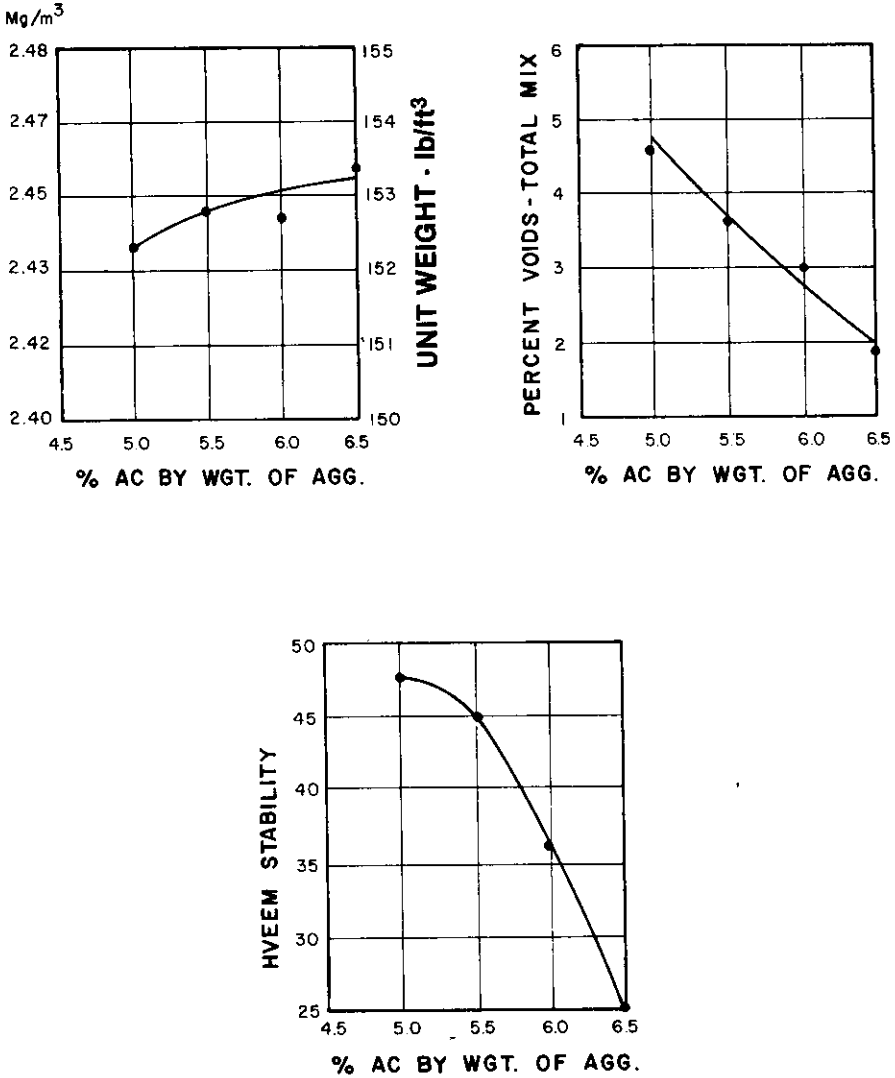


Figure 4.26 **Test Property Curves for Hot-Mix Design Data by the Hveem Method (Refer to Figure 4.25).**

In the application of these requirements, the optimum asphalt content for the mix design should be the highest percentage the mix will accommodate without reducing stability or void content below minimum values.

The optimum asphalt content can be determined from the outline shown in Figure 4.27. It utilizes stabilometer values, percent air voids, and observations of surface flushing or bleeding of specimens after compaction.

Table 4.6 Hveem Design Criteria

Traffic Category	Heavy		Medium		Light	
Test Property	**min.**	**max.**	**min.**	**max.**	**min.**	**max.**
Stabilometer Value	37	—	35	—	30	—
Swell	less than 0.762 mm (0.030 in.)					

NOTES:

1. Although not a routine part of this design method, an effort is made to provide a minimum percent air voids of approximately 4 percent.
2. All criteria, and not stability value alone, must be considered in designing an asphalt paving mix.
3. Hot-mix asphalt bases that do not meet these criteria when tested at 60° C (140° F) are satisfactory if they meet the criteria when tested at 38° C (100° F) and are placed at 100 mm (4 in.) or more below the surface. This recommendation applies only to regions having a range of climatic conditions similar to those prevailing throughout most of the United States. A different lower test temperature may be considered in regions having more extreme climatic conditions.
4. Traffic Classifications:

Light:	Traffic conditions resulting in a Design EAL $<10^4$.
Medium:	Traffic conditions resulting in a Design EAL between 10^4 and 10^6.
Heavy:	Traffic conditions resulting in a Design EAL $>10^6$.

The following steps outline this procedure to select the optimum asphalt content:

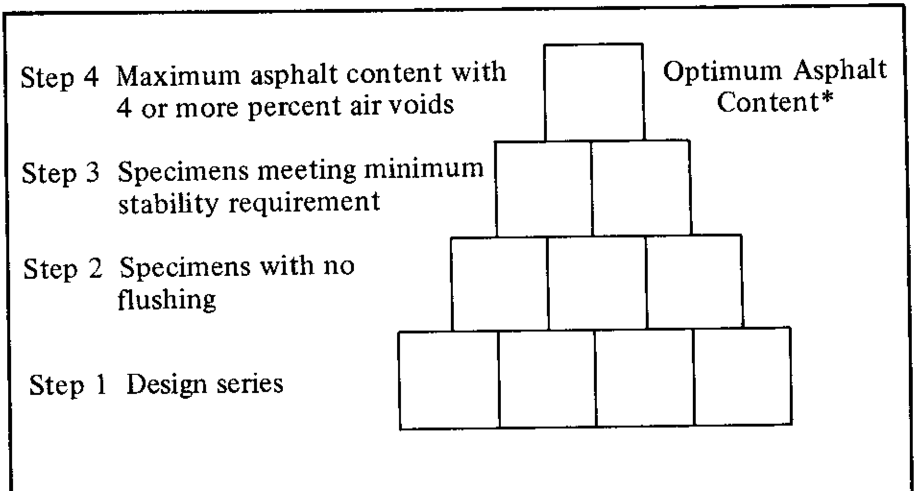
(a) Using Figure 4.27, insert in Step (1) of the pyramid, the asphalt contents used for preparing the series of mix design specimens. Insert asphalt contents in order of increasing amounts from left to right with the maximum asphalt content used in the square on the right.

(b) Select from Step (1) the three highest asphalt contents that do not exhibit moderate or heavy surface flushing and record on Step (2). (Surface flushing and/or bleeding is considered "Slight" if the surface has a slight sheen. It is considered "Moderate," if sufficient free asphalt is apparent to cause paper to stick to the surface but no distortion is noted. Surface flushing is considered "Heavy" if there is sufficient free asphalt to cause surface puddling or specimen distortion after compaction).

(c) Select from Step (2) the two highest asphalt contents that provide the specified minimum stabilometer value and enter them in Step (3).

(d) Select from Step (3) the highest asphalt content that has at least 4.0% air voids and enter in Step (4).

(e) The asphalt content in Step (4) is the optimum asphalt content. However, if the maximum asphalt content used in the design set (Step 1) is the asphalt content entered on Step (4), additional specimen must be prepared with increased asphalt content in 0.5 percent increments and a new optimum asphalt content determination made.


Step 4 Maximum asphalt content with 4 or more percent air voids
Optimum Asphalt Content*
Step 3 Specimens meeting minimum stability requirement
Step 2 Specimens with no flushing
Step 1 Design series

*Optimum asphalt content is not valid if the maximum asphalt content used in the design series (Step 1) is the asphalt content arrived at in Step 4. In this event additional specimens must be prepared with increased asphalt content in 0.5 percent increments and a new analysis made.

Figure 4.27 Procedures for Selecting Optimum Asphalt Content, Hveem Method of Design.

Chapter 5
Manufacture of Hot-Mix Asphalt

Hot-mix asphalt is produced at a hot-mix asphalt facility. A hot-mix facility is an assembly of mechanical electronic equipment where aggregates are blended, heated, dried and mixed with asphalt to produce a hot asphalt mix meeting specified requirements. The facility may be stationary (located at a permanent location) or it may be portable (moved from job to job). In general, every facility can be categorized as either (1) a batch facility, or (2) a drum-mix facility. The differences between the two facilities are described later in this chapter. An understanding of the fundamentals presented in this chapter should enable the reader to adapt this knowledge to almost any hot-mix asphalt operation.

These facilities are also used for manufacturing asphalt mixes from emulsified and cutback asphalt. Normally they are produced with less heat and partially dried aggregates. These mixes have many of the same quality controls imposed upon them as do hot mixes.

Particular attention should be given to the adjustments made at the facility to maintain control over the flow of materials. Testing and record keeping requirements of the plant inspector are also important considerations.

This chapter is divided into two sections. The first introduces the reader to the various aspects of mixing facilities and discusses some of the many variations. The second section highlights some of the necessary checks and controls needed to operate and inspect such facilities. To accomplish this, Section 5.2 repeats some of the descriptive material from Section 5.1. It is hoped that this redundancy will not be overly distracting.

SECTION 5.1 Overview of a Hot-Mix Asphalt Facility

☐ Introduction
☐ Aggregate Storage and Cold Aggregate Feed
☐ Aggregate Drying and Heating
☐ Hot Aggregate Storage
☐ Mineral Filler
☐ Asphalt Storage
☐ Batch Mixing Facility
☐ Drum-Mix Facility

SECTION 5.2 Operation and Inspection of Hot-Mix Asphalt Facilities

☐ Introduction
☐ Materials Storage
☐ Cold Aggregate Feeding
☐ Drying and Heating
☐ Screening and Grading
☐ Temperatures
☐ Batch Facility Operations
☐ Drum Mix Facility Operations
☐ Miscellaneous Inspection Duties
☐ Sampling and Testing

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2. *Principles of Construction of Hot-Mix Asphalt Pavements,* MS-22, Asphalt Institute.
3. *Asphalt Cement Content Diagnostic Approach for Hot Mix Asphalt Facilities, QIP-109,* National Asphalt Pavement Association.
4. *Safety Procedures for the Prevention of Fires and Explosions in Hot-Mix Asphalt Facilities, HS-7,* National Asphalt Pavement Association.
5. *Safe Storage and Handling of Hot Asphalt, IS-180,* Asphalt Institute

TRAINING AIDS

1. *Introduction to Hot-Mix Asphalt Manufacturing Facilities,* TAS-18, Slide-tape presentation, National Asphalt Pavement Association.

5.1 Overview of a Hot-Mix Asphalt Facility

Introduction

Asphalt paving mixes made with asphalt cement are prepared at an asphalt mixing facility. Aggregates are blended, heated and dried, and mixed with asphalt cement to produce a hot asphalt paving mixture. There are two types of mixing facilities currently in widespread use. They are the batch mixing facility and the drum-mixing facility.

Operations of a batch mixing facility include:

(a) cold aggregate storage
(b) proportioning
(c) drying/heating
(d) screening
(e) hot aggregate storage
(f) measuring and mixing of aggregate and asphalt
(g) discharge into truck, storage silo or surge bin

Aggregate is removed from storage, or stockpiles, in controlled amounts and passed through a dryer where it is dried and heated. The aggregate then passes over a screening unit that separates the material into different size fractions and deposits them into bins for hot storage. The aggregate and mineral filler, when used, are then withdrawn in controlled amounts, combined with asphalt, and thoroughly mixed.

Operations of a drum-mixing facility are:

(a) cold aggregate storage
(b) proportioning
(c) heating and mixing of aggregate and asphalt
(d) storage silo

These processes are discussed later in this chapter.

Types of Hot-Mix Asphalt Facilities

Asphalt hot-mix facilities, whether batch or drum-mix type, can also be classified as stationary or portable. The stationary facility (Figure 5.1) is permanently situated and is not normally dismantled and moved. The portable facility (Figure 5.2) can be disassembled, moved by rail or highway, and reassembled with a minimum of time and energy.

In the batch-type mixing facility, different size fractions of hot aggregate in storage bins are withdrawn in desired amounts to make up one batch for mixing. The entire combination of aggregate is then dumped into a mixing chamber called a pugmill. The asphalt, which has also been weighed, is thoroughly mixed with the aggregate. After mixing, the material is emptied from the pugmill in one batch.

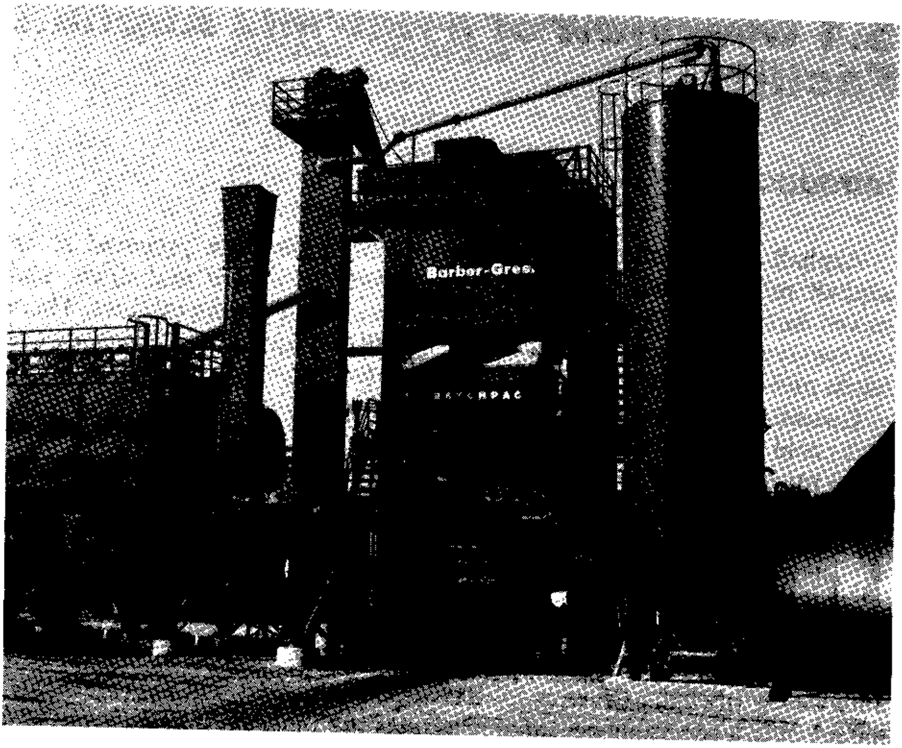


Figure 5.1 Batch Mixing Facility.

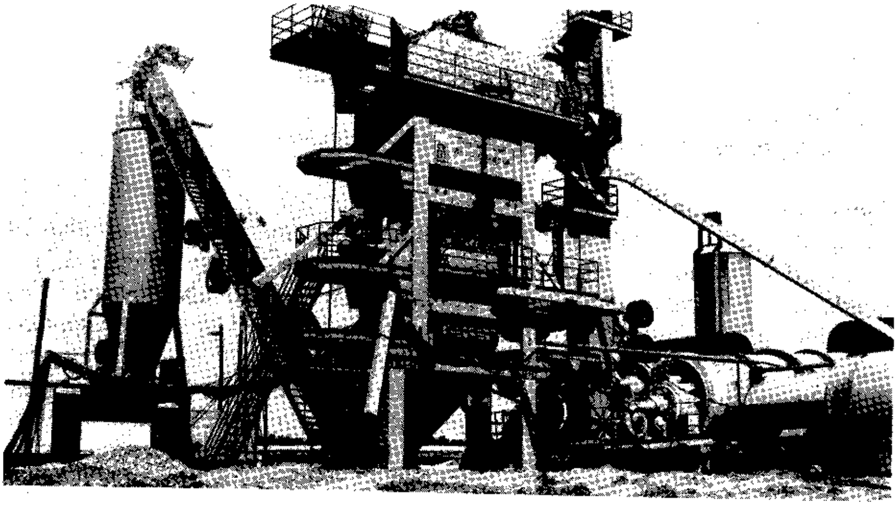
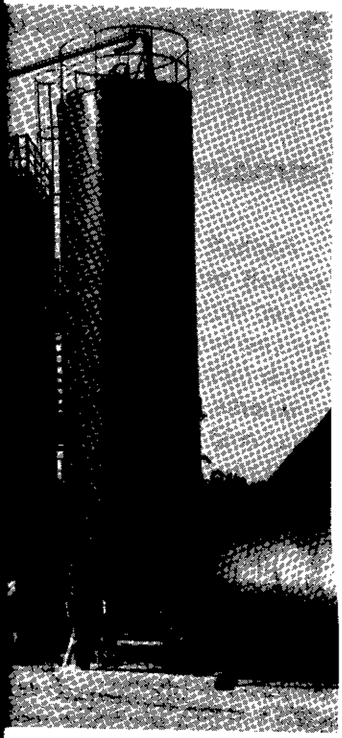
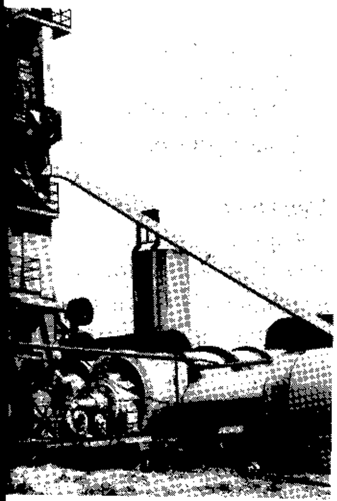


Figure 5.2 Batch Mixing Facility, Portable Type. *(Courtesy C.M.I.)*





. (Courtesy C.M.I.)

In a drum-mixing facility, hot asphalt mixtures are produced using a cold feed system, an asphalt proportioning system, a rotating drum mixer, and a storage silo. Figures 5.3, 5.4 and 5.5 show various types of drum-mixing facilities. In this process, aggregate is both heated and blended with asphalt in the same unit; the drum mixer.

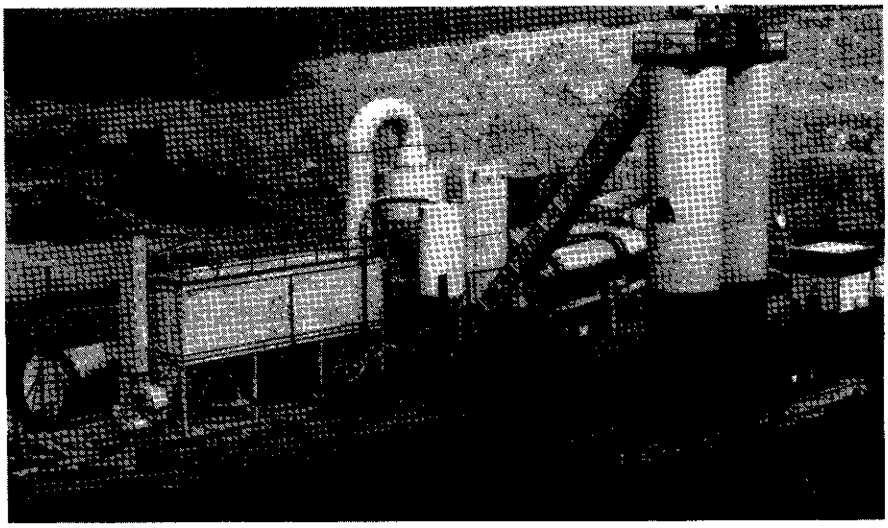


Figure 5.3 Stationary Drum-Mixing Facility. *(Courtesy ASTEC Industries Inc.)*

Figure 5.4 Portable Drum-Mixing Facility.
(Courtesy Standard Havens Products, Inc.)

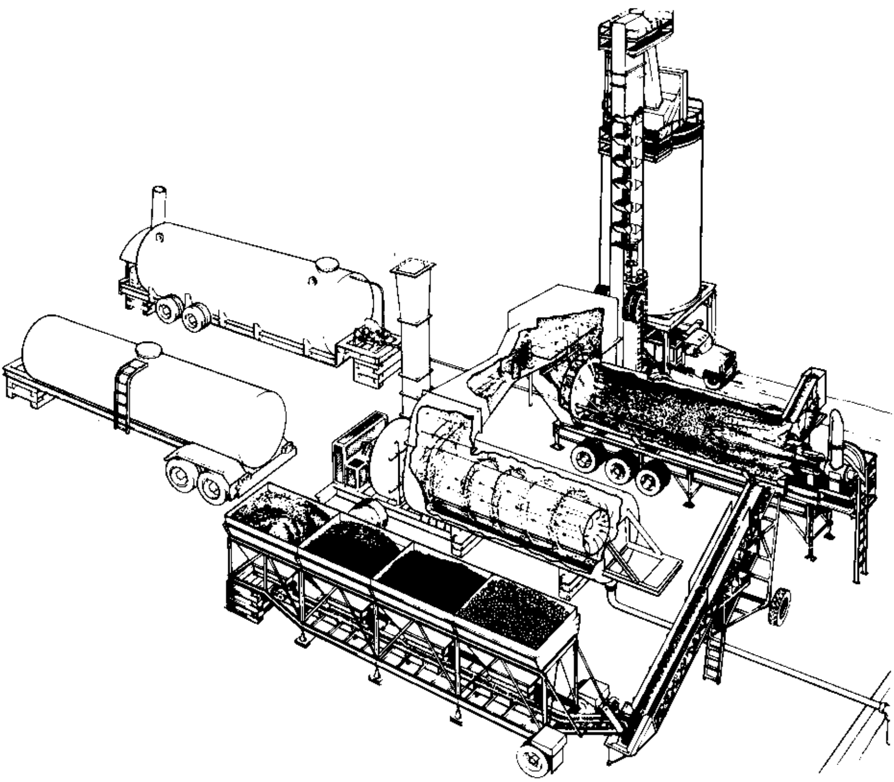


Figure 5.5 Drum-Mixing Facility Schematic *(Courtesy Barber-Greene Co.)*

Site Location

Factors that must be considered in selecting a site location include ease or convenience of access and egress, surface drainage, adequacy of area to accommodate materials storage needs and a facility layout conducive to the safe and efficient operations of aggregate handling equipment, trucks delivering asphalt and aggregates or trucks hauling hot-mix away. Consideration must also be given to the proximity to residential and business development that may be adversely affected by noise, dust and heavy traffic.

Aggregate Storage and Cold Aggregate Feed

Aggregate Storage

A storage area is set aside for stockpiling aggregates. Stockpiles should be carefully constructed on clean, stable surfaces with ample provision made to prevent intermingling or other contamination of the materials.

Vehicle movement to and from the stockpiles should be completely unencumbered. Sometimes it is inadvisable to store more than a few days supply of material near the plant. A smooth operation requires orderly delivery of materials to storage, and an equally orderly removal of materials from storage at about the same time.

As mineral filler is subject to caking or hardening from moisture, it is handled differently than other aggregates, and separate storage is provided to keep it protected from dampness.

Cold Feed Supply

Cold aggregate feed is the first major component of the hot-mix asphalt facility. The cold feeder may be charged by one or a combination of the following methods:

1. Open-top bins with two to five compartments, usually fed by a crane with a clamshell bucket or a front-end loader.

2. Tunnel under stockpiles separated by bulkheads. Materials are stockpiled over the tunnel by belt conveyor, truck, crane, or front-end loader.

3. Bunkers or large bins. These usually are fed by trucks, car unloaders, or bottom-dump freight cars emptying directly into the bunkers.

Types of Feeders and Controls

Aggregate feeder units are located beneath the storage bins or stockpiles, or in positions that ensure a uniform flow of aggregate. Feeder units have controls that can be set to produce a uniform flow of aggregate to the cold elevator (Figure 5.6). There are several types of feeds, including continuous belt (Figure 5.7), vibratory (Figure 5.8), and apron flow (Figure 5.9). Generally, belt feeders are best for accurate metering of fine aggregates. Coarse aggregates usually flow satisfactorily with any of these feeders.

For a uniform output of the hot mix, input of materials must be accurately measured. The importance of feeding the exact amounts of each size aggregate into the dryer at the correct rate of flow cannot be overemphasized.

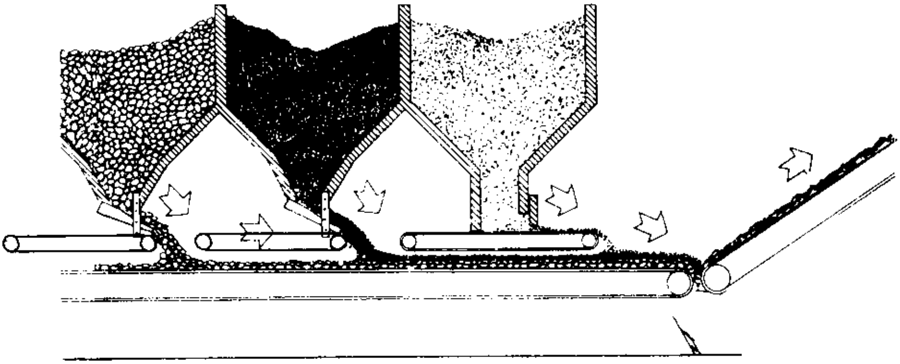


Figure 5.6 Three-Bin Cold Feeder and Belt.

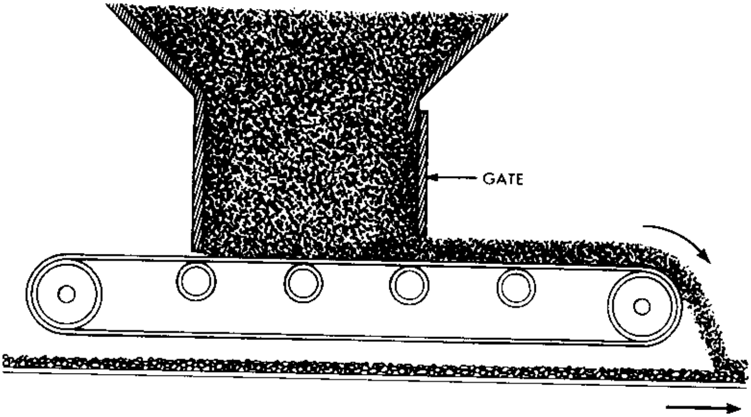


Figure 5.7 **Continuous Belt Feeder.**

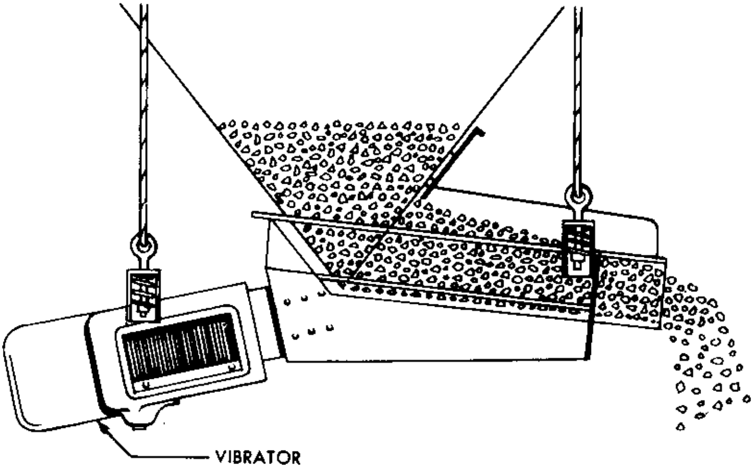


Figure 5.8 **Electromagnetic Vibratory Feeder.**

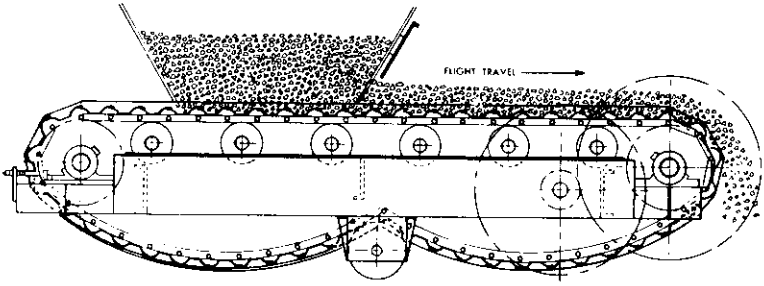


Figure 5.9 Apron Flow Feeder.

Aggregate Drying and Heating

Aggregate Heating

One of the basic units in all facilities is some device to dry and heat the aggregate. At a batch facility this unit is the dryer and, at the drum-mixing facility, a drum mixer. These units are a necessary part of the hot-mix operation for they dry and heat aggregates coming from the cold feed supply, thus making them suitable for mixing with asphalt.

The dryer at a batch facility is usually a large rotating metal drum mounted at a slope and equipped with a gas or oil heating unit at the lower end (Figure 5.10). The air flow moves in a direction opposite that of the aggregate (the counter-flow principle). This arrangement results in fines (dust) being carried out of the aggregate blend by the dust collection system. Hot gases from the burner pass from the lower end of the rotating drum and out the upper end. Cold aggregate is fed into the upper end of the dryer and is picked up by steel angles or flights mounted on the inside. As the dryer drum rotates, the aggregate is lifted up and dumped through the hot gases and flame. Because of the inclination, the aggregate also gradually works its way toward the lower end of the dryer. The hot aggregate then discharges from the lower end onto a hot elevator that carries it to the screens and hot storage.

The design and operation of the drum mixer at a drum-mixing facility is quite different. The aggregate heating and drying operation takes place in the drum-mixer (Figure 5.11). Additionally, asphalt cement is added and mixed with the heated aggregate while still in the drum-mixer. These procedures and modifications to the drum-mixing operation will be discussed later in this section.

Drying is the most expensive operation in mix production. It is also the most frequently encountered bottleneck in the plant operation. The best dryer is one that meets a desired performance level at the lower investment and operating cost.

Temperature Indicating Device

Specifications normally designate the temperature of the aggregate for mixing. Aggregate temperature is measured by either a thermometer or a thermocouple attached to an indicating pyrometer. Pyrometers react much faster to changes in temperature and are therefore preferred. Specifications frequently require a record of temperatures be made and preserved, and for this, recording devices are installed.

The Dust Collector

The dust collector (Figures 5.12 and 5.13) is generally operated adjacent to and in conjunction with the dryer, and is necessary for efficient plant operation. The collector eliminates or abates the dust nuisance that might result from exhaust air from the dryer. Modern dust-collection systems such as the baghouse using fabric filters are highly efficient. Provisions are usually made in the dust collector to return the collected dust back to the hot

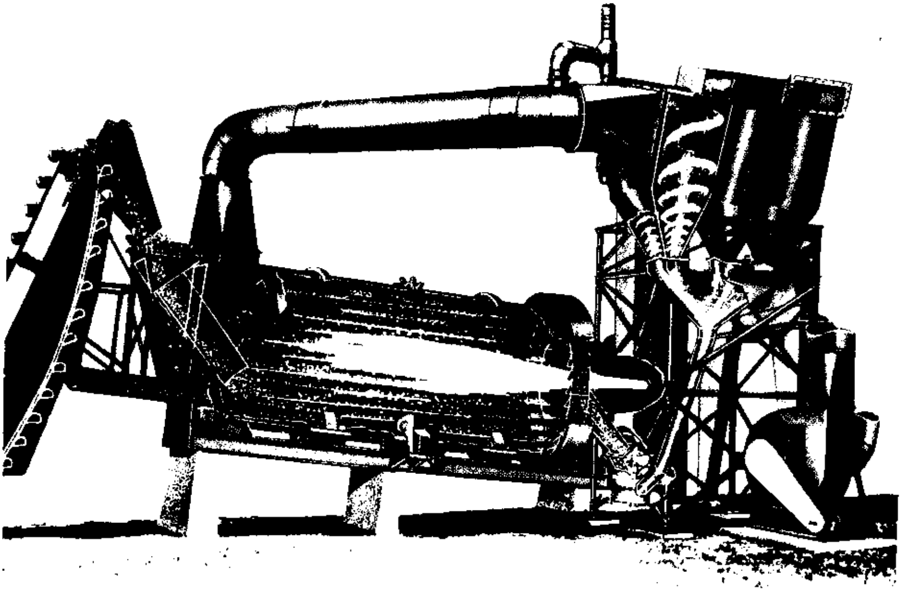


Figure 5.10 Dryer at a Batch Mixing Facility.

aggregate as it emerges from the dryer or drum mixer. If the collected dust is not needed or unsuitable for use in the asphalt mixture, it may be removed from the collector and wasted. Two or more of these devices may be included in a dust collection system, which then contains a primary collector and one or more final, or secondary, collectors.

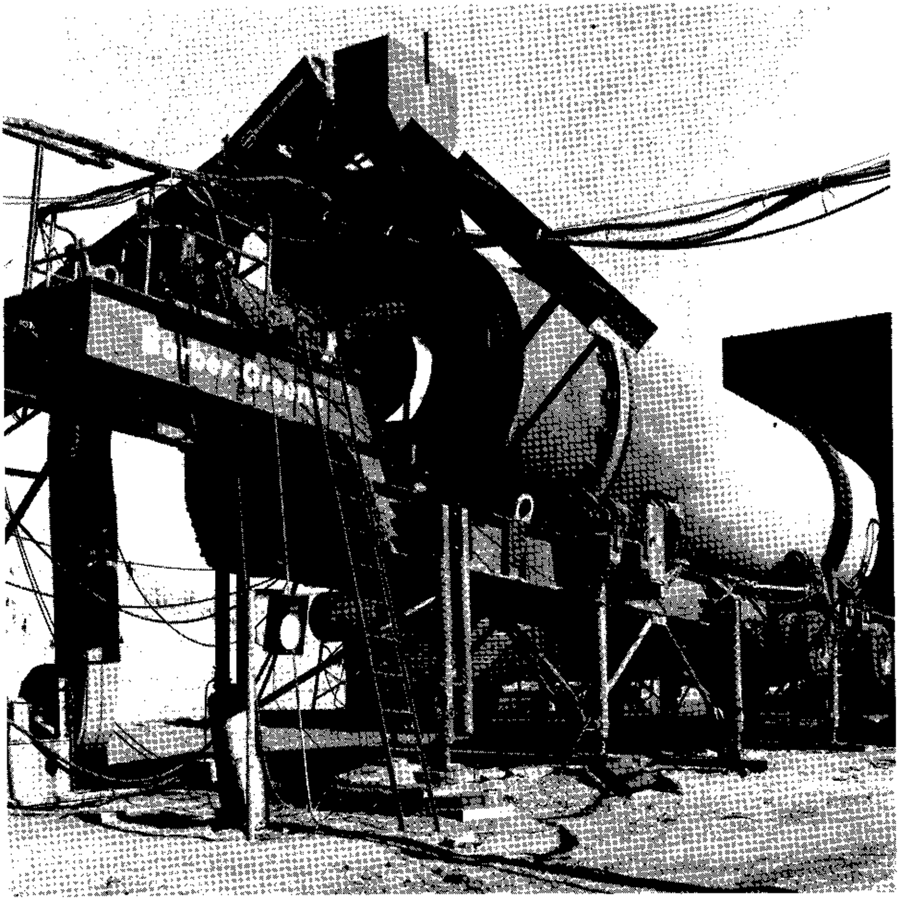
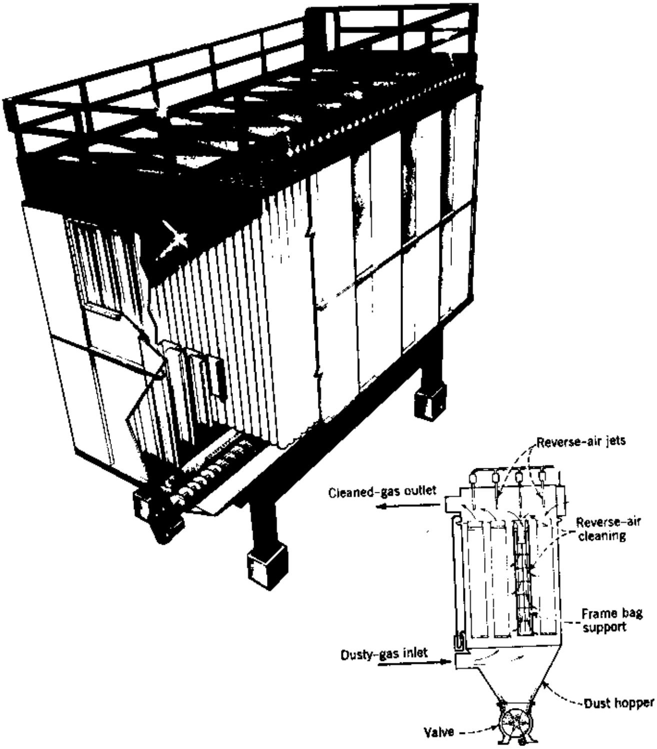


Figure 5.11 Drum-Mixer.



(Side-View
Showing Fabric
Filter or Bags)

Figure 5.12 Typical Baghouse Dust Collector.
(Courtesy Standard Havens Products Inc.)

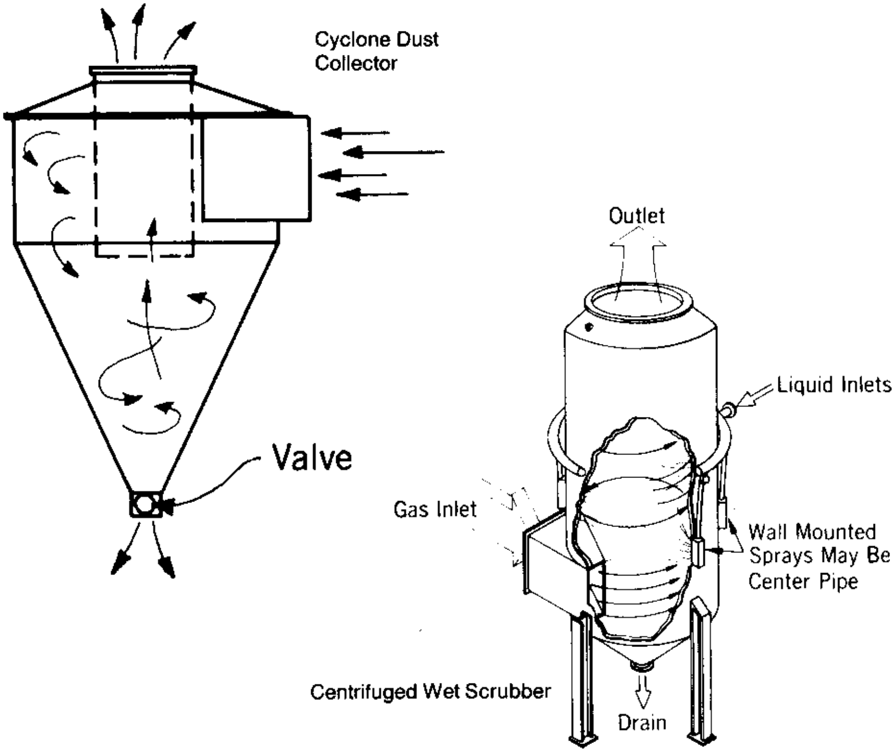


Figure 5.13 Other Dust Collection Systems.

Hot Aggregate Storage - Batch Mixing Facility

Screening Unit

Dried aggregates in a batch mixing facility are transported from the dryer by an enclosed hot elevator and deposited onto a unit which contains screens, storage bins, and aggregate proportioning devices. Screens are used to separate the aggregate into fractions of specified size and deposit each into a separate bin, Figure 5.14. An imbalance in the hot bins under the screens signals a need for corrective action elsewhere—usually in the cold aggregate feed.

Screen sizes selected are usually based on separating the aggregate into equal percentages of material by weight. The smallest practical size, however, is a 3.35 mm (No. 6) screen. To separate aggregate into specified sieve sizes used in testing, screen cloths having slightly larger openings are used.

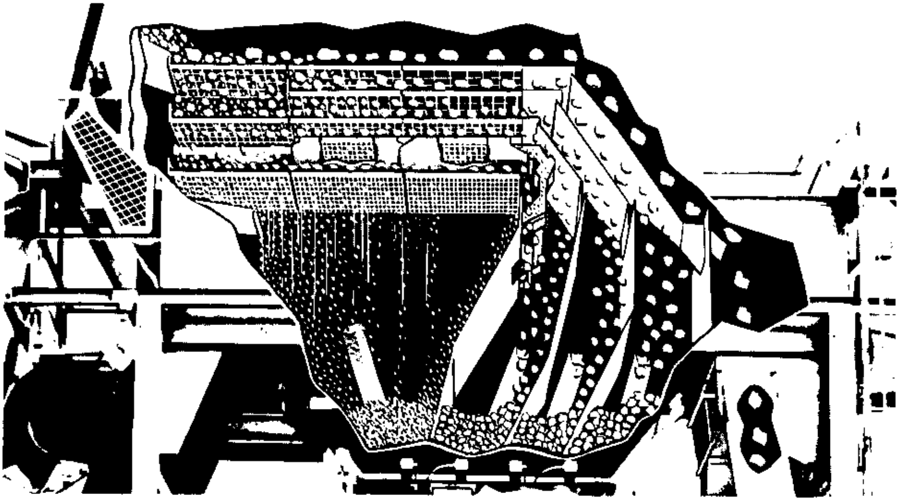


Figure 5.14 Cutaway View Showing Details of Flow of Material Through Screens and Bins. *(Courtesy Iowa Manufacturing Company)*

Hot Bins

Hot bins are used to temporarily store heated and screened aggregate in the various size fractions required. Each bin should be large enough to prevent depletion of the material when the mixer is operating at full capacity. Each bin should have an overflow pipe to prevent aggregate from backing up into the other bins. The overflow flow pipe also prevents overfilling to the point where the vibrating screen will ride on the aggregate. Should this happen it would result in a heavy carryover and probable damage to the screens.

Material is withdrawn from the hot bins in predetermined proportions and at a specified rate. If the level of aggregate in hot storage has little variation during facility operation, a balanced flow of aggregate is being achieved.

Mineral Filler

Commercial Filler

Some hot-mix asphalt manufacturing facilities have separate feeding systems for introducing commercial mineral filler into the mix. The mineral filler is deposited in a special ground-mounted feeder, and with a screw conveyor, dust-tight elevator, and overflow chute is in turn deposited in a surge hopper. From here it is added to the aggregate as it is withdrawn from the hot bins for mixing.

Collected Dust

If the material gathered in the dust collector can be recombined satisfactorily with the aggregates in the mix, some or all of it may be returned to be used as filler.

A continuously turning worm screw at the bottom of the dust collector removes the dust and fines that have settled in the collector chamber and may deposit them at the bottom of the hot elevator where the aggregate emerges from the dryer. It is preferable, however, that they be collected and metered / weighed into the mixing chamber, just as commercial mineral filler would be handled.

Asphalt Storage

Asphalt Tank

Asphalt storage at the facility should be sufficient to maintain uniform operation while still allowing leeway for delayed shipments and testing time.

Asphalt storage tanks should be calibrated so the amount of material remaining in the tank can be determined at any time. Tanks should be equipped with a suitably placed recording thermometer having a 24-hr minimum time range.

Asphalt Heating and Circulation

Since asphalt must be fluid enough for movement through the delivery and return lines, it must be heated. Heating may be done by the circulation of steam or hot oil through coils in the tank, or it may be done electrically.

All storage tanks, transfer lines, pumps and weigh buckets should have heating coils or jackets to maintain the asphalt at the required temperature. Return lines discharging into the storage tank should be submerged below the asphalt level in the tank at all times to prevent the asphalt from oxidizing (Figure 5.15). To break the vacuum in the lines when the pump is reversed, two or three vertical slots should be cut in the return line within the tank, but above the high-level mark of the stored asphalt.

Batch Mixing Facility

Up to this point there have been few differences and some similarities between the kinds of asphalt mixing facilities. From now on, distinction will be made as to the type of facility being described, batch type or drum mix type.

Briefly, in a batch-type facility the aggregate is drawn from hot-bin storage in predetermined amounts and dropped into a pugmill mixer in batches. The correct amount of asphalt is then added and mixed with the aggregate. The mixture is then dropped into a truck (Figure 5.16), or transferred to a surge bin or storage silo.

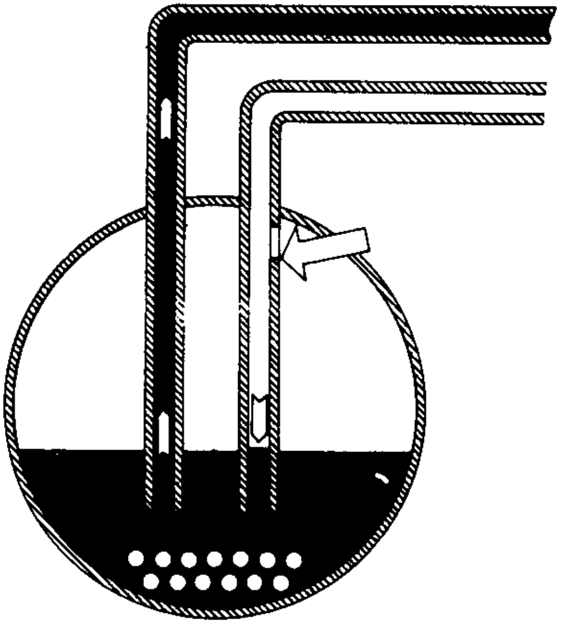


Figure 5.15 Asphalt Return Line. Large Arrow Points to Vertical Slot.

Figure 5.16 Hot-Mix Asphalt Being Loaded Directly From the Pugmill.

Aggregate Weigh Hopper

Aggregates are released from the hot bins into the weigh hopper, beginning with the largest size aggregate and progressing down to the finest size, with the mineral filler added last. The amount from each bin is determined by the batch size and the proportion required to be blended with the material from the other bins to meet a job mix formula.

The weigh hopper is suspended from scale beams and the amounts of aggregate weighed cumulatively.

Before withdrawal starts there should always be sufficient material in the hot bins for a complete batch. If a bin is starving or is running over, chances are that an adjustment in the cold feed is required.

Asphalt Bucket and Meter

Asphalt may be weighed in a special bucket or it may be measured by a meter for each batch. When weighed into a batch, asphalt is pumped into a bucket of known weight and weighed on a scale.

When metering devices are used, a volumetric measurement is made. The volume of asphalt changes with temperature. Some asphalt meters have built-in temperature-compensating devices that correct the flow of asphalt when changes in temperature occur. The volume of asphalt pumped between two meter readings may be weighed as a means of calibrating the meter.

Pugmill

A twin shaft pugmill-type mixer is commonly used in all modern batch mixing facilities (Figure 5.17) where it is mounted directly beneath the weigh box and asphalt bucket, and high enough so that it may discharge the mixture into the truck or other hauling unit.

Mixing

When aggregates are withdrawn from the hot bins as described earlier, some dry mixing takes place as the materials are deposited in the weigh hopper, as well as when they are deposited in the pugmill. The wet mixing time begins with the start of the flow of asphalt from the bucket or meter.

Asphalt film on the aggregate is hardened by exposure to air and heat. The mixing time should be no longer than necessary to get a uniform distribution of aggregate sizes and a uniform coating of asphalt on all aggregate particles. Speed of the mixer shafts, condition of the paddles, and the arrangement and pitch of the paddles are factors governing mixing efficiency.

Upon completion of the mixing, the bottom of the pugmill mixer opens and discharges the contents, which are dropped directly into a truck or conveyed to a storage silo.

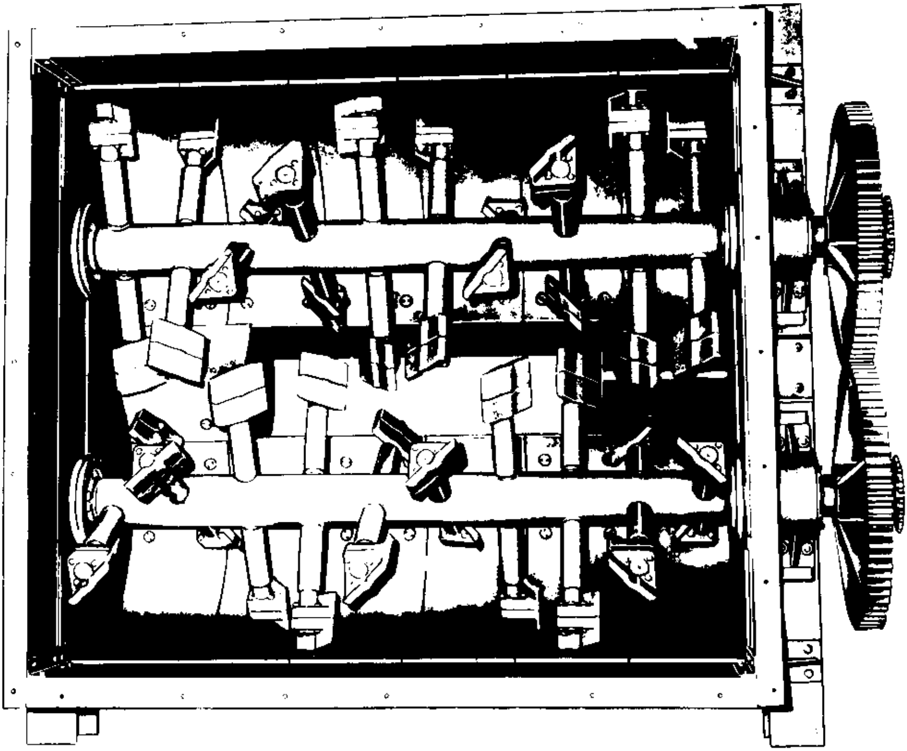


Figure 5.17 Pugmill Mixer for a Batch Facility. *(Courtesy Iowa Manufacturing Company)*

Hot Storage Silo

A hot storage silo is used to store temporarily the production of hot mix before being hauled. It is a cylindrical shaped structure where hot mix is fed in at the top and, as needed, loaded into trucks from the bottom (Figure 5.18).

Automated Mixing Facilities

Batch facilities fall into three categories, depending on the degree of automation: manual, semiautomatic and automatic. Modern facilities are usually fully automatic and computerized.

One thing nearly all these facilities have in common is power control of the weighing and mixing process. Even in manual facilities, air or hydraulic cylinders actuated with electric switches have replaced hand lever systems. They operate supply bin gates, feeders, asphalt valves, the weigh box discharge gate, and the pugmill discharge gate.

In the semi-automatic facilities, all operations from weigh box discharge to pugmill discharge are under automatic cycle control. These include operation of the weigh hopper discharge gate, dry mixing (when required),

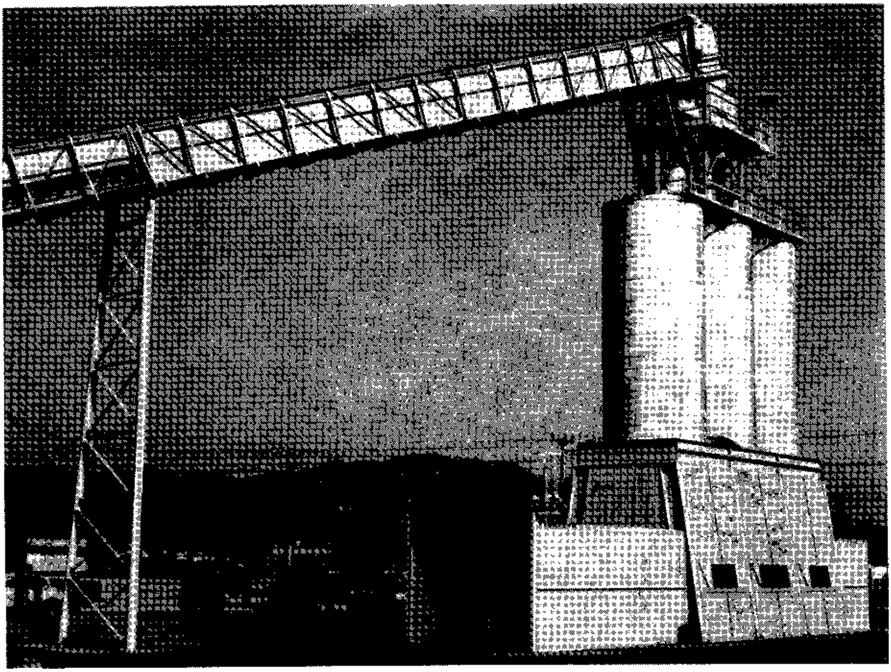


Figure 5.18 Large-Capacity Insulated Asphalt Concrete Storage Silos.

asphalt weigh bucket, wet mixing, and operation of the pugmill discharge gate. Limit switches ensure that all functions occur in the proper sequence.

Computerized controls provide completely automatic control of all the facility functions, and afford automatic record keeping as well as continuous inventory of materials and production. A modern computerized control system also includes automatic burner control and remote cold feed total and proportional control. Such a system enables a qualified operator to remotely control the entire plant operation (Fig. 5.19).

Drum-Mix Facility

Basic Facility

Simply stated, drum mixing is a process in which hot asphalt mixtures are produced in a plant without hot aggregate screens, hot bins, and pugmill mixer (Fig. 5.20). The basic plant consists of a cold feed system, a rotating drum mixer with modified flights, an asphalt proportioning and dispensing system, a dust collection system and a surge bin.

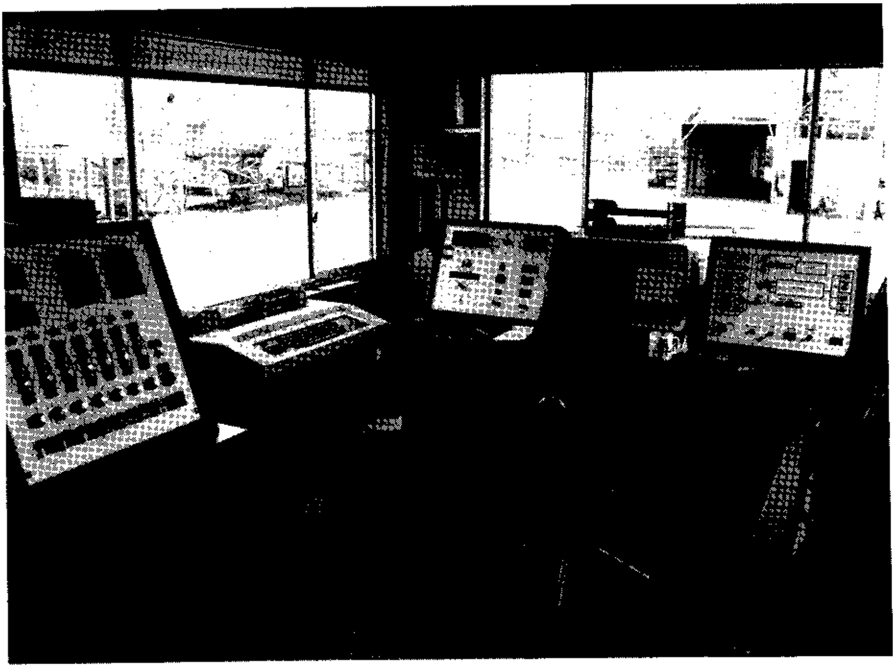


Figure 5.19 Control Station for Automated Facility.

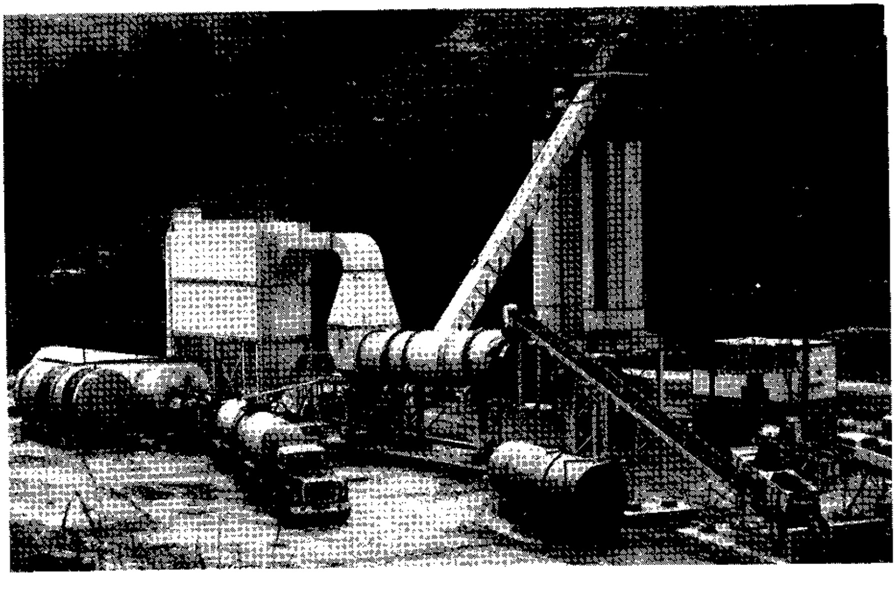


Figure 5.20 Drum-Mixing Facility.

Drum Mixer

The heart of the drum-mix facility is the mixer itself. It may at first glance appear to be a conventional batch plant rotary dryer, however, the drum-mixer not only dries aggregate but also blends the aggregate and asphalt into a hot mixture. Thus the drum-mixer at a drum-mixing facility is quite different in its function and operation (and therefore in design) from the dryer at a batch facility. One primary design difference is that the heating unit (flame) is at the upper end of the drum. The cold aggregate enters the drum at the flame end. The aggregate and the heated air both flow in the same direction (referred to as *parallel flow*) rather than in the opposite direction (referred to as *counter-flow*), as they do in a batch plant dryer. The drum mixer operation and other design differences will be discussed later in this section.

Controls

Additionally, the following controls and equipment are required for this type of plant to ensure a quality product that meets specifications in all respects:

1. Separate cold feed controls, for each aggregate size.
2. Interlocking of cold feed and asphalt and additives when used.
3. Sensors to determine the moisture content of aggregate so adjustments in proportions of materials can be made if necessary.
4. Means for sampling all materials components while the plant is in full production.
5. Automatic burner controls.
6. Primary dust collector that can feed back collected material.
7. Sensors to measure temperature of the hot mixture at discharge.

Operation

Aggregate gradation control is achieved in the crushing and stockpiling operations. Accurately-controlled feeders proportion the aggregate as it leaves the cold bins. Belt scales that continuously weigh and monitor the combined aggregates are interlocked with a metering asphalt pump to maintain a constant aggregate-to-asphalt ratio.

Aggregates enter the drum, where heat from the burner located at the same end dries and heats them. As it moves through the drum, asphalt is added and aggregates and asphalt are thoroughly blended. Continued convection drying also occurs. The mixture of hot asphalt and moisture released from the aggregate produces a foaming mass that traps the fine material (dust) and aids in the coating of the larger particles.

Some drum-mix facility variations include counter-flow of heated air and mid-point recycled aggregate entry. The line drawing, Figure 5.21, shows how the concept works. The virgin aggregate enters one end of the drum and moves toward the burner (or counter to the flow of the hot gas stream). Note the burner in this variation is located in the drum at the opposite end from the aggregate entry. The asphalt cement and recycled asphalt material (RAP

- see Chapter 12) are introduced at a point behind the burner flame—in an area where the materials are protected from high heat and exposure to exhaust gases.

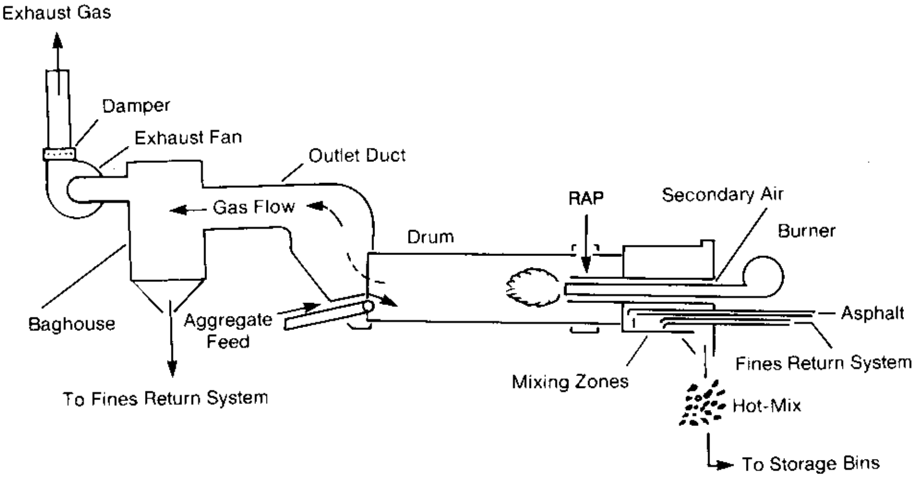


Figure 5.21 (a) Counter Flow Drum Mixer.

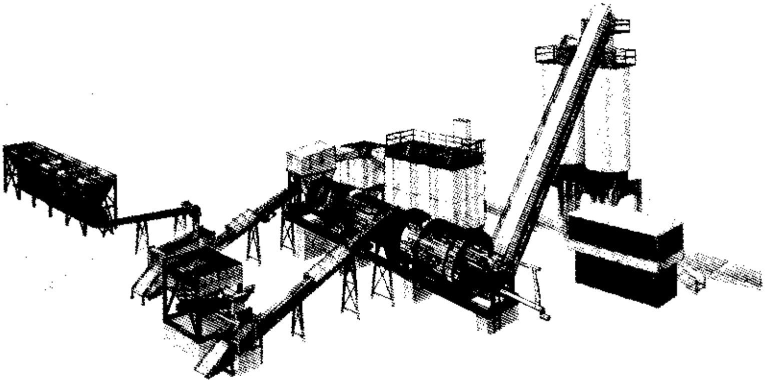


Figure 5.21 (b) Facility With Counter Flow Drum Mixer. *(Courtesy Standard Havens)*

Other drum-mixing variations include a separate asphalt "coater unit" shown in Figure 5.22. In this instance, the drum mixer is now only drying aggregate, similar to the dryer function in the batch facility. The mixing of aggregate and asphalt takes place at a different location—the coater unit.

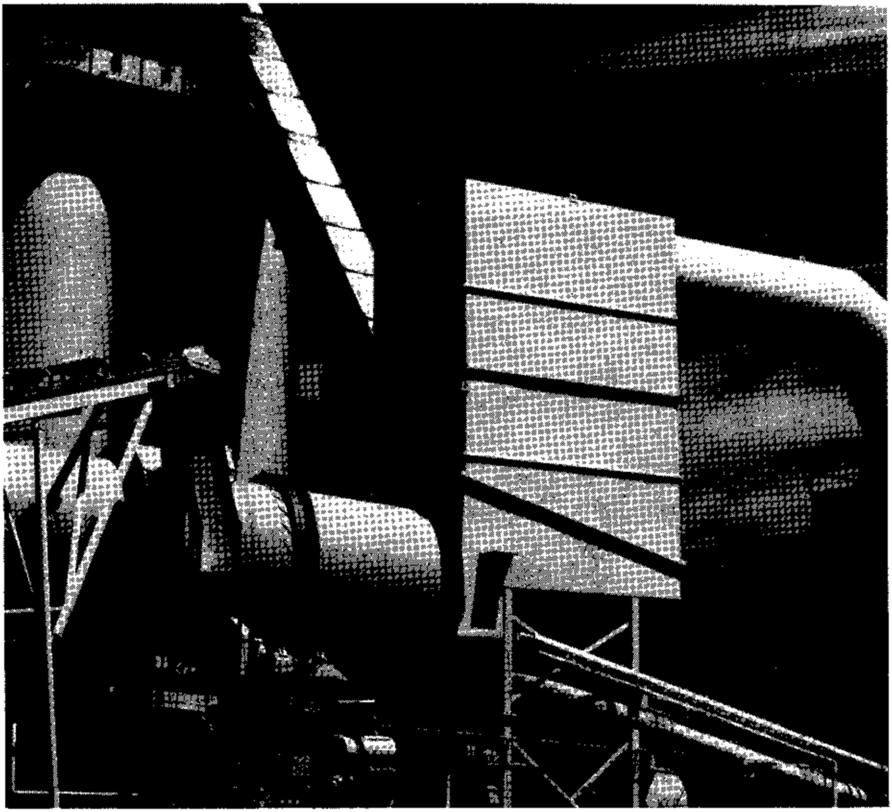


Figure 5.22 Asphalt Coater. *(Courtesy ASTEC Industries Inc.)*

Storage of Hot-Mix Asphalt

Most modern hot-mix asphalt facilities are equipped with surge or storage bins. Such capability is absolutely essential in the continuous operation of a drum-mixing facility. A surge bin is connected to the plant by a conveying system and is intended to hold mix for relatively short time periods. Generally, it is not insulated because holding time is expected to be only a matter of hours. A storage bin is similar to a surge bin except that it is insulated because it is intended to hold the hot mix for longer storage periods.

The most popular shape of surge or storage bins is cylindrical with a conical section at the bottom. A variety of conveying systems are in use, *including: belt conveyors, bucket elevators, skip hoists, screw conveyors, and slat conveyors (Figure 5.23). Studies indicate that the bins can be* charged without segregation or an appreciable drop in temperature.

Surge systems offer several benefits because stop-and-go operation of the facility is minimized. Variability in mix composition and temperature associated with start up and stopping is reduced by more continuous plant operation. Also, the emission of extra air pollutants each time the facility starts up is minimized. Productivity is increased by running continuously during normal working hours and not just when trucks are available for loading. Also, facilities can begin producing mix earlier in the morning, before roadway conditions permit the placing of the hot mix.

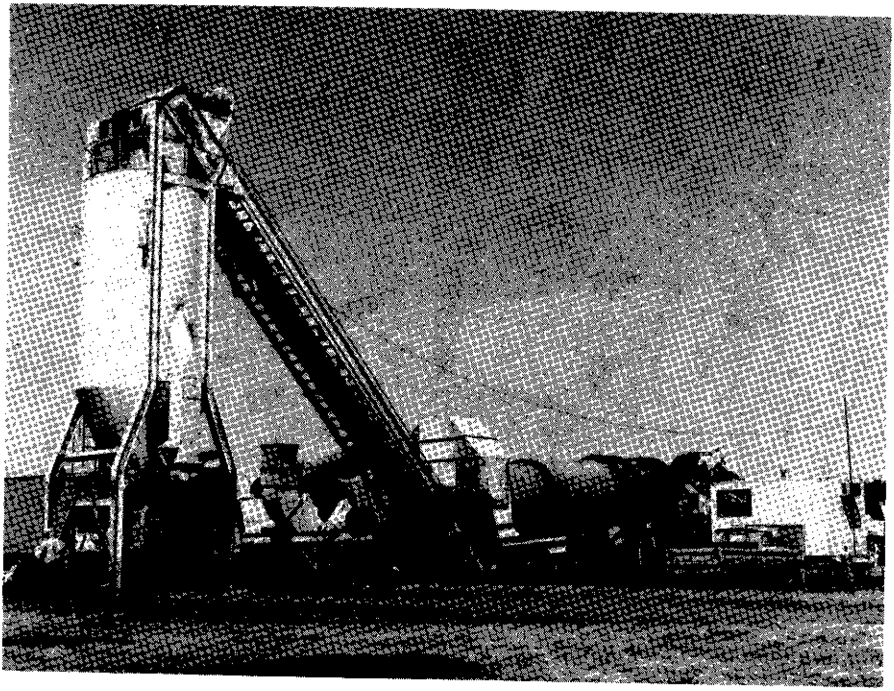


Figure 5.23 Mix Storage at Drum-Mixing Facility.

5.2 Operation and Inspection of Hot-Mix Asphalt Facilities

Introduction

Engineer-Contractor Cooperation

It is vital for the engineer and contractor to cooperate fully if high-quality construction is to be realized. The engineer represents the owner who, for most paving projects, is the traveling public. He must make sure that the project reflects a high standard of workmanship throughout each construction phase.

The engineer, his inspectors and technicians should be thoroughly acquainted with asphalt mixing facilities and their operation. Neither the engineer nor inspector should make any adjustment to the facility operation, but they may advise the operating superintendent if adjustments are needed. In the same vein, the superintendent and his technicians should not make adjustments that would effect a change in the final product without first consulting the engineer or inspector.

Continuity of Operations

One of the essentials for a consistent and high-quality hot-mix asphalt is continuous plant operation. One of the essentials for a consistent and uniform asphalt pavement is a continuous paving operation. Quality of the mix, workmanship, or both, suffer when one or the other operates intermittently. Should the paving operation be unavoidably interrupted for some reason other than plant production, the facility operation will have to be interrupted unless some other provision is made.

Balancing Facility Production

A hot-mix asphalt facility may be rated by its potential for producing asphalt paving mix in tonnes (tons) per hour. Frequently it is judged on the basis of its pugmill size alone. The hourly rate is then calculated from the mixer size and the required mixing time.

However, it is important to recognize the interdependence of other facility components, such as cold feed, dryer, screening unit, and hot bins. They should be of sufficient size and capacity to provide a continuous and adequate flow of materials. Limiting factors may be environmental conditions, aggregate characteristics, or mix requirements. For best results, components should be selected on the basis of anticipated conditions under which the facility will be operated.

The aggregate drying operation is the most frequently encountered bottleneck. This operation is a vital role in overall facility performance. The percentage of moisture to be removed has a major effect on the required capacity of the dryer or drum-mixer.

Another capacity limiting factor, this one at a batch facility, is the screening unit. Screens can only sort the material at a given rate. When loaded beyond this rate, smaller-sized grain particles are carried over into another bin. This is termed a carry-over.

The other components also have their limitations and they should be recognized. Rated production is limited to the production limit of the component having the smallest capacity.

Materials Storage

Aggregate Stockpiles

Aggregate are generally hauled to the site while the facility is being erected. The technician, at this early stage, sees that a sufficient number of samples of proper size are taken. These samples must be representative of the materials delivered.

It is the responsibility of the inspector to see that aggregates are stored and handled in a manner which minimizes degradation, segregation and contamination.

Suitable storage areas must be provided for each size aggregate. There must be no intermingling of different aggregates through overlapping or faulty bin partitions.

Aggregate stockpiles should be constructed in layers. The thickness of the layer depends upon the material and how it is placed. If a truck haul is used, the loads can be dumped closely together over the entire surface; the size of the truck load governs the thickness of the layer.

When stockpiling with a crane, each bucket load of material should be deposited (not cast) adjacent to another so that the thickness of the stockpile layer is uniform over the entire area. When chutes or conveyors are used, baffles or perforated chimneys may be needed to limit segregation in certain materials.

Building a conical stockpile by freefall of aggregate from a chute or belt conveyors lets the fine materials blow away or causes the larger particles to roll down the sides, resulting in segregation of the aggregate sizes. This type of stockpile construction should be avoided whenever possible.

Mineral Filler Storage

Mineral filler may arrive at the mixing facility in bulk shipments or in bags. In either case, the material must be protected from moisture.

Asphalt Storage

The facility site should be arranged so that the asphalt supply is readily accessible to the mixer. A ramp should be furnished near the storage tank, on which the transports can be placed in an inclined position while unloading so that the asphalt drains completely.

Asphalt quantities stored at the site must be sufficient to allow uniform operation, even when delayed shipments and testing time are taken into account. Most facilities have at least two asphalt tanks—one working tank and one or more storage tanks (Figure 5.24). Where more than one grade of asphalt is required for a job, at least one tank will be needed for each grade.

Asphalt storage tanks should be calibrated so that the amount of material remaining in the tank can be determined at any time. They also must be heated to keep the asphalt fluid enough to move through the delivery and return lines. Heating is done electrically or by circulating hot oil or steam through coils in the tank. Regardless of the heating method used, an open flame must never come in contact with the tank.

Where circulating hot oil is used, the oil level in the reservoir of the heating unit should be checked frequently. A drop in the level could indicate leakage of the hot oil into the tank, leakage which results in contamination of the asphalt.

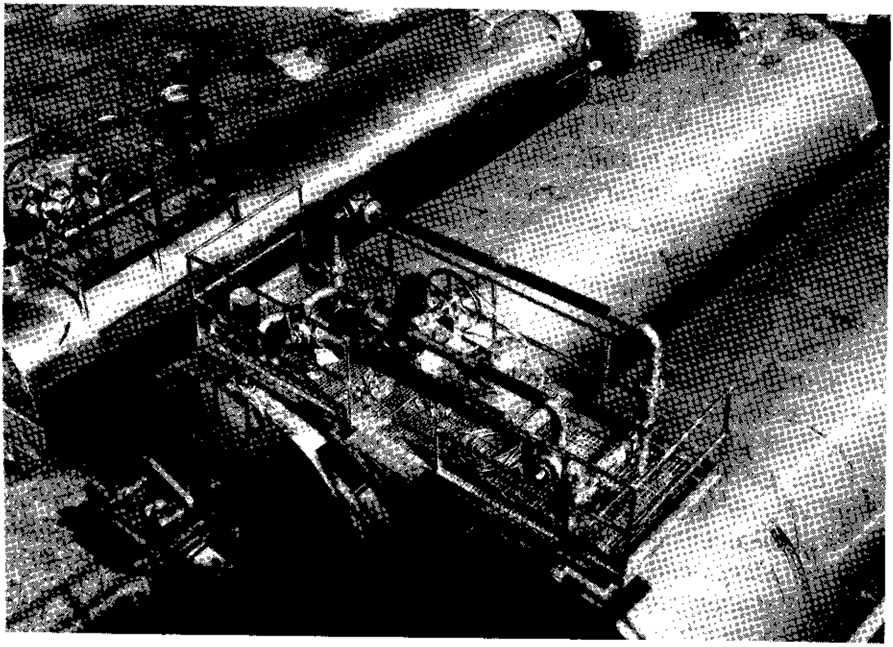


Figure 5.24 A Series of Asphalt Storage Tanks.

All transfer lines, pumps and weigh buckets also must have heating coils or jackets so that the asphalt will remain fluid enough to pump. One or more thermometers should be placed in the asphalt feed system to ensure control of the asphalt temperature.

Return lines discharging into the storage tank should be submerged below the asphalt level in the tank at all times to prevent the asphalt from oxidizing during circulation. To break the vacuum in the lines when the pump is reversed, and to clear the lines, two or three vertical slots must be cut in the return line within the tank, but above the high level mark of the stored asphalt.

A valve or a spigot may be installed in the circulating system to allow sampling of the asphalt. When sampling from the circulating system, exercise extreme care, as pressure in the lines may cause the hot asphalt to splatter.

Cold Aggregate Feeding

Loading Cold Feed Storage Units

When charging the cold bins, care should be taken to minimize aggregate segregation and degradation. This can be prevented by taking the same precautions as for proper stockpiling. Enough materials should be maintained in all bins to provide a positive and uniform flow.

If cold aggregate is stockpiled over a tunnel and belt, care should be taken when handling material over the feeders. For example, bulldozers used on stockpiles may cause segregation and degradation. Mud and dirt or other objectionable material should not be tracked onto the stockpile by these machines. Vibration from a bulldozer can cause fine particles in the coarse stockpile to filter down into a layer that later is pushed to the feeder. The result may be an imbalance of the feed. This may be minimized by varying the path of the bulldozer to the feeder. Also, continuous abrasive action by particles being moved about can cause degradation in some types of aggregates.

If the stockpile level above the tunnel is maintained by a dragline or clamshell, the operator must be careful not to pick up material from the same position in the stockpile in successive withdrawals.

When a front-end loader is used, the operator should not pick up material from the storage stockpile at ground level. The scoop should be held high enough above the ground to prevent contamination when filling.

When picking up material from a stockpile, care should also be taken to avoid taking materials from areas that are excessively wet (such as near the bottom of the pile), or even unusually dry areas. Excessive fluctuations in the moisture content of the aggregate being fed to the dryer will result in unacceptable fluctuations in the mix temperature—which in turn will result in changes in the mix placement and compaction characteristics and will adversely affect the service performance of the finished pavement. Strive for uniformity in all aspects of all mixing operations.

If trucks are used to charge the bin, they should deposit their loads directly above the feeder.

When the stockpile is replenished by overhead belts or elevating conveyors, the flow of materials should be controlled by baffles installed within the bin.

Calibrating and Setting Feeders

The cold aggregate feeder gates should be calibrated. Manufacturers may furnish approximate calibrations for gate openings of their equipment, but the only *accurate* way to set the gates is to prepare calibration charts for the gates, using the aggregates employed in the mix.

To calibrate a feeder gate, the gate is set at three or more different gate openings and the feeder is operated at each setting. When the feeder is running normally, the material for a number of revolutions is measured into a container. The flow of material per minute for that gate opening may be determined:

$$q = \frac{W}{r}(1 - w) \tag{1}$$

where

q = dry weight of aggregate, kg (lb) per min;
W = weight of aggregate measured, kg (lb);
r = number of revolutions for measurement; and
w = moisture content of aggregate

This formula may be adapted for apron feeders, reciprocating plate feeders, and belt feeders.

When the gates discharge onto a belt conveyor, output may be checked by closing all gates except one, which is set at one of the calibration points. If the gates cannot be closed completely, it may be necessary to stop the feeder, or disconnect it if it is mechanically driven. The cold feed is started and brought to normal operating speed. Then it is stopped and the material from a measured section of the belt is removed and weighed. The amount of material delivered per minute for that gate opening is determined by using Equation 1 and letting:

r = length of belt section where material was removed, mm (ft); and
R = belt speed, mm (ft) per min.

When this has been done for all gates that feed aggregates to the dryer, a calibration chart may be prepared. The gate opening in millimetres (inches) is plotted on the horizontal scale and the dry weight in kg (lb) per minute plotted on the vertical scale (Figure 5.25). The gate opening may then be determined for the desired rate of flow, which may be determined:

Metric	Customary
$q=0.167\ TP$	$q=0.333\ TP$
where	where
q = required rate of flow, kg/min	q = required rate of flow, lb/min
T = plant production, tonnes/hr	T = plant productions, tons/hr
P = percent by weight of total mix	P = percent by weight of total mix

For example, a batch facility is producing 200 tonnes (220.5 tons) of asphalt concrete per hour. Material from cold bin No. 1 is supplying the aggregate that makes up 37 percent of the mix. The required rate of flow from bin No. 1 is:

Metric	Customary
$q = 0.167\ (200)\ (37) =$ 1232 kg/min (dry weight)	$q = 0.333\ (220.5)\ (37) =$ 2716 lb/min (dry weight)

Drying and Heating (Batch Facility)

Dryer Operation

Most dryers are designed for average moisture content. Very wet aggregate (Figure 5.26) reduces dryer capacity and requires corrective measures; for example:

(1) The amount of heat can be increased by burning more fuel while the flow of aggregate remains constant, or

(2) The aggregate can be held in the dryer longer by changing the slope of dryer or by modifying the flights inside the dryer (i.e., adding "kick-back" flights).

There is a limit to the increase in heat that is possible without exceeding exhaust draft capacity. Beyond that limit the draft must be increased or the rate of aggregate flow must be reduced. In very humid areas, or when aggregates are exceptionally wet, the contractor may operate two dryers in tandem or run the aggregate through the same dryer twice.

Most problems in dryer operation are caused by crowding more material through the dryer than it can properly handle. However, there are other factors that affect efficient operation. Several factors involve the burner. If an oil burner is used, it is important that the proper grade of fuel be used. The oil must be properly atomized by the blower. The velocity of the draft air, which combines with the atomized fuel oil for combustion, must be in balance with the blower air and the amount of fuel oil being fed into the burner. If the blower air, draft air, and flow of fuel oil are not in balanced

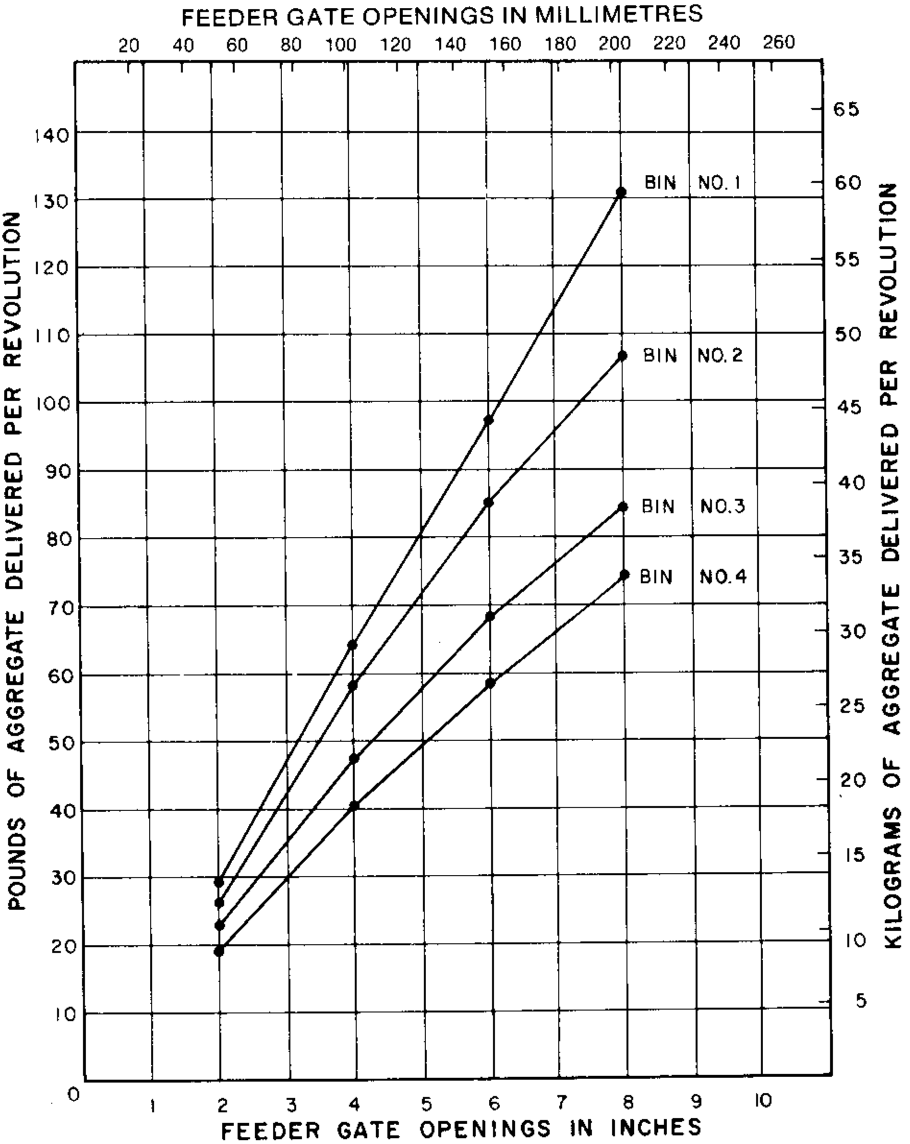


Figure 5.25 Calibration Chart for Cold Bin Feeds.

adjustment, it may cause incomplete combustion of the fuel, leaving an oily coating on the aggregate particles that may affect the mixture. Black smoke from the exhaust stack indicates that the oil is not being completely burned.

Lack of balance between blower air and draft air velocities can also cause back pressure within the dryer. This creates a "puff back" at the burner end. Puff back indicates that the draft is not sufficient to accommodate the air pressure being introduced by the burner blower. The solution is either to increase the draft air or to reduce the pressure of the blower air.

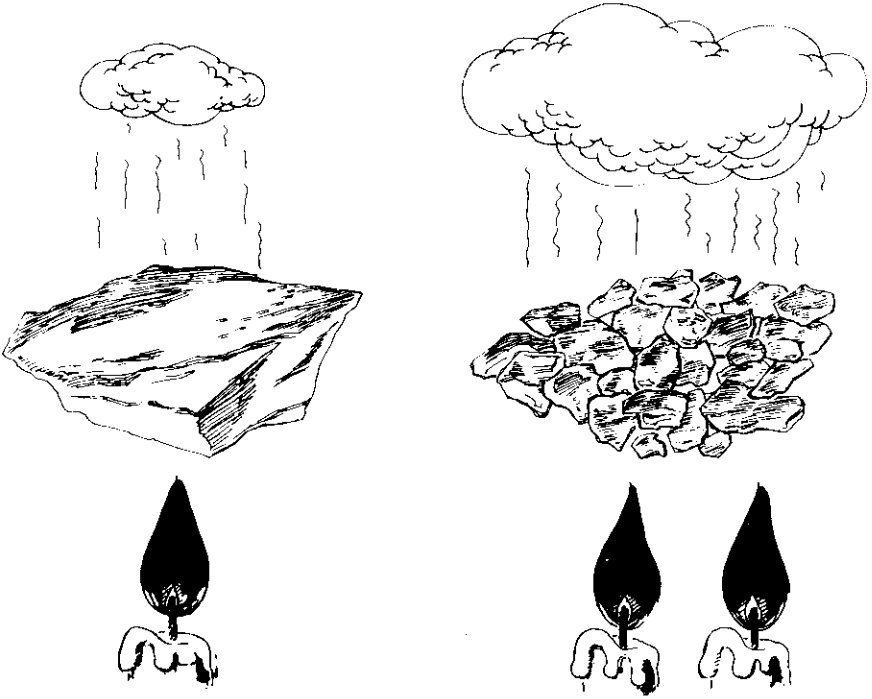


Figure 5.26 **Fine Aggregate Will Hold More Moisture Than Coarse Aggregate, Requiring More Heat for Drying.**

Dryers having natural gas or liquid petroleum gas burners rarely develop combustion problems. However, an imbalance between the gas pressure, combustion air, and draft still can occur. The gas burner must be the correct type for the gas pressure available.

Collected Dust

If the material removed from the dust collector can be recombined satisfactorily with the aggregates in the mix, some or all of it may be returned to the facility. The amount returned depends on the combined grading of the finished mix. If the collected dust is unsuitable, prohibited by mix specifications, or more than can be used, it should be diverted to a storage bin for disposal or other use. If collected (baghouse) fines are permitted in the finished mix, the same quantity must be used in the laboratory in the development of the job mix formula.

Screening and Grading (Batch Facility)

Screen Size and Capacity

Hot screens are used to separate aggregate into specified sizes. The hot bins splits can be determined by a plot of the total aggregate specification. Splits should be made so that the amount of material in each bin is in balance. The 3.35 mm (No. 6) slotted size, however, is the smallest practical size that can be used. For a four-bin split (Figure 5.27), the material larger than the 3.35 mm (No. 6) sieve is proportioned into the other three bins. To separate aggregate into specified sieve sizes used in testing, screen cloths having slightly larger openings are used on the asphalt plant vibrating screen (See Table 5.1).

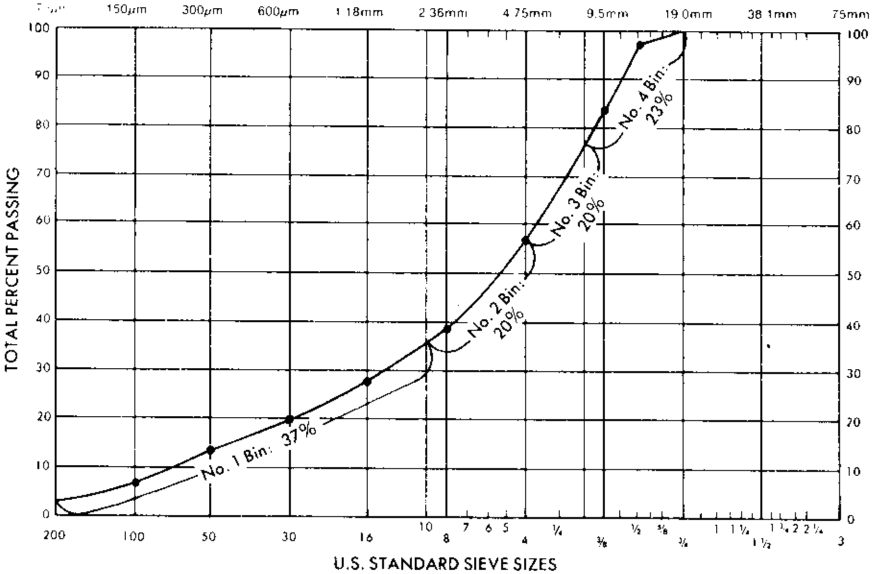


Figure 5.27 Selecting Size Splits for 4-Bin Storage.

Screen deck arrangements are such that the smallest particle sizes fall into the first bin and the larger particles into the later bins. A $3^1/_3$ deck screen arrangement is shown in Fig. 5.28, where the actual size of the square openings of the vibrating screens are also shown. The sizes shown are those used to divide the aggregate gradation as illustrated in Figure 5.27.

The tonnes (tons) per hour processed per square metre (foot) of screen area will vary because screen capacity is dependent on type of screen, aggregate, and specification requirements.

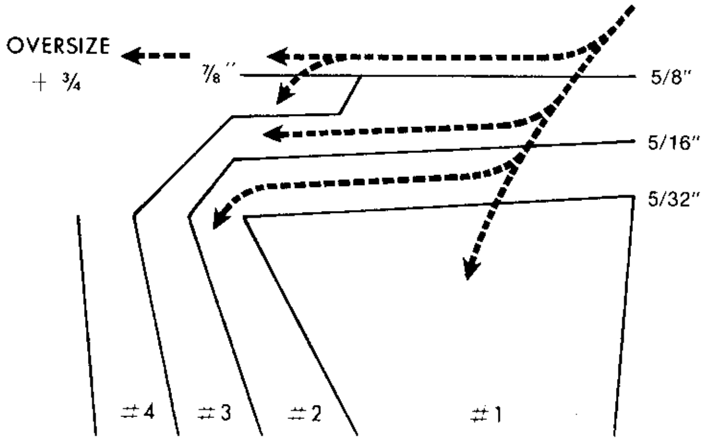


Figure 5.28 3-1/3 Deck Screen Arrangement for 4-Bin Hot Storage.

Table 5.1 Screen Selection Table

U.S. Std. Sieve Sizes	Actual Size of Square Openings of Vibrating Screens	U.S. Std. Sieve Sizes	Actual Size of Square Openings of Vibrating Screens
3.35 mm (No. 6)	3.96 mm (5/32 in.)	16.0 mm (5/8 in.)	19.0 mm (3/4 in.)
4.00 mm (No. 5)	4.76 mm (3/16 in.)	19.0 mm (3/4 in.)	22.4 mm (7/8 in.)
4.75 mm (No. 4)	6.3 mm (1/4 in.)	22.4 mm (7/8 in.)	25.0 mm (1 in.)
6.3 mm (1/4 in.)	8.0 mm (5/16 in.)	25.0 mm (1 in.)	28.6 mm (1-1/8 in.)
8.0 mm (5/16 in.)	9.5 mm (3/8 in.)	31.5 mm (1-1/4 in.)	34.9 mm (1-3/8 in.)
9.5 mm (3/8 in.)	11.2 mm (7/16 in.)	38.1 mm (1-1/2 in.)	41.3 mm (1-5/8 in.)
11.2 mm (7/16 in.)	12.5 mm (1/2 in.)	45 mm (1-3/4 in.)	50 mm (2 in.)
12.5 mm (1/2 in.)	14.3 mm (9/16 in.)	50 mm (2 in.)	57 mm (2-1/4 in.)

Table 5.2 is a general guide for determining screen capacity. It may be used as a guide for dried aggregates, and where no screening problems will be encountered.

Screen Efficiency

The conditions and cleanliness of the screen will, to a great extent, control their efficiency. If the effective screening area is reduced by plugged screen openings, or if more material is fed to the screens than they can handle, carry-over will result.

Excessive wear of the screen wire and/or tears in the screen results in enlarged openings and holes (Figure 5.29). This puts oversize material in the bin. Usually, a small amount of oversize material does no harm, provided the amount does not fluctuate excessively or exceed specification requirements.

Table 5.2 Vibrating Screen Capacity

Screen Size Clear Opening	Tonnes/hr/m² (Tons/hr/ft²)	Tonnes/hr (Tons/hr) 0.91 m × 2.44 (3' × 8')	1.22 m × 3.05 m (4' × 10')	1.22 m × 4.27 m (4' × 14')
4.76 mm (3/16 in.)	20 (2)	45 (50)	73 (80)	100 (110)
6.3 mm (¼ in.)	29 (2½)	54 (60)	91 (100)	127 (140)
9.5 mm (⅜ in.)	32 (3¼)	73 (80)	118 (130)	163 (180)
12.5 mm (½ in.)	37 (3¾)	82 (90)	136 (150)	191 (210)
16.0 mm (⅝ in.)	42 (4¼)	91 (100)	154 (170)	209 (230)
19.0 mm (¾ in.)	46 (4¾)	104 (115)	172 (190)	240 (265)
22.4 mm (7/8 in.)	51 (5¼)	113 (125)	191 (210)	268 (295)
25.0 mm (1 in.)	56 (5¾)	127 (140)	209 (230)	290 (320)

Note: The values in this table are neither the maximum nor minimum that may be expected. The capacity will be affected by many factors, including aggregate moisture content, gradation of the material, screen load vs. amount passing, and size of screen wire. Screens produced for use in U.S. are manufactured only to customary units.

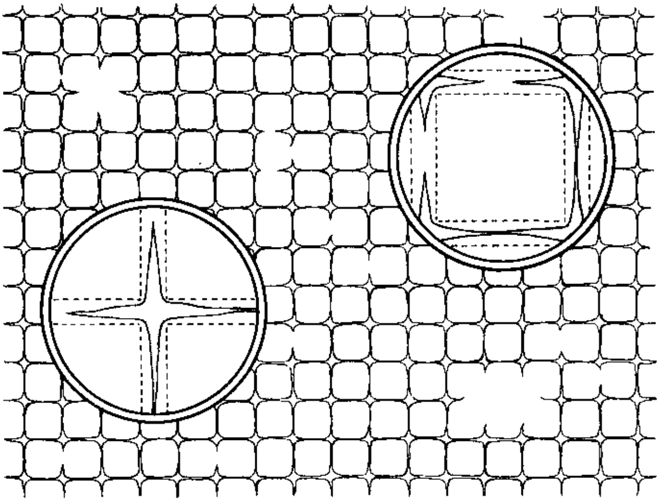


Figure 5.29 Screen Wear.

Carry-Over

When screen openings are plugged, or when too much material is fed to the screens, many particles that should pass through ride over the screens and drop into a bin designated for a larger size. Carry-over, then, is the depositing of finer particles in a bin meant for larger, aggregate particles.

Varying carry-over causes a lack of uniformity in the aggregate gradation and in the mixture. Excessive carry-over results in an increase in the amount of fine aggregate in the total mix, thus increasing the surface area to be coated with asphalt. Excessive carry-over, or its fluctuations, will be apparent from the sieve analysis made from the contents of the individual hot bins. Corrective measures include cleaning screens, regulating the quantity of material coming from the cold feed, or a combination of both.

This carry-over condition is experienced more often where fine screening area (No. 1 bin) is limited. Some carry-over is permitted in normal screening, however, and the permissible amount in each bin is usually specified. Adjustments must be made in the amount of material pulled from each bin in order to compensate for the carry-over.

Daily visual inspection of the screens for cleanliness is recommended, preferably before starting the day's operations. When conditions warrant, the screens should be cleaned.

Hot Bins

The hot bins should be large enough to prevent depletion of material when the mixer is operating at full capacity. Their partitions should be tight, free from holes, and high enough to prevent intermingling of aggregates. Each hot bin should be equipped with an overflow pipe to prevent aggregate from backing up into other bins. The overflow vents should be checked frequently to make sure they are free-flowing. When intermingling occurs, the specified gradation is destroyed.

Sometimes the fines fraction of the aggregate hangs up in corners of the bin. When this build-up releases, it results in an excess of fines in the mix. This rush of fine material normally occurs when aggregate in the bin is reduced to a low level. The condition can be corrected by welding fillet plates in the bin corners to eliminate the 90-degree angles.

Other potential obstacles to a good mix include shortages of material in one bin or an excess in another; worn gates at the bottom of the bins that allow leakage of aggregate into the weigh hopper; and sweating of the bin walls.

Bin shortages or excessives are corrected by adjusting the cold feed. For example, if the minus 3.35 mm (No. 6) (No. 1 bin) is overflowing while it is necessary to wait for the plus 8 mm ($^{5}/_{16}$ in.) (No. 3 bin), the opening of the cold bin supplying most of the minus 3.35 mm (No. 6) material should be decreased slightly, and the bin supplying most of the plus 4.76 mm ($^{3}/_{16}$ in.) material should be opened slightly. If the coarse bin is overflowing while the other two remain at a satisfactory level, there is too much total material coming in. Thus, the cold bin supplying most of the aggregate to the coarse bin should be reduced slightly.

It is not a good practice to make two adjustments at once. For example, if it is known that the total feed is deficient and also that one bin is running a little heavy, it is better to adjust the total feed first and then make a new adjustment on the relative feed from the various bins.

Sweating occurs when moisture vapor in the aggregate and in the air condenses on the bin walls. This usually happens only at the beginning of the day's operation or when the coarse aggregate is not thoroughly dried. Sweating may cause dust to accumulate. When released suddenly, the dust adds unwanted fines to the mix.

Bin Sampling

Modern hot-mix asphalt facilities are equipped with devices for sampling hot aggregate from the bins. These are designed to divert the flow of aggregate from the feeders, or gates, under the bins into sample containers. The best place to obtain a sample is from the bin gates as the material falls into the weigh hopper. It is essential that sampling facilities be constructed and located so that the samples will be representative of the material in the bins.

In the flow of material over the plant screens, particularly in the No. 1 bin, fine particles fall to the near side of the bins and coarse particles to the far side (Figure 5.30).

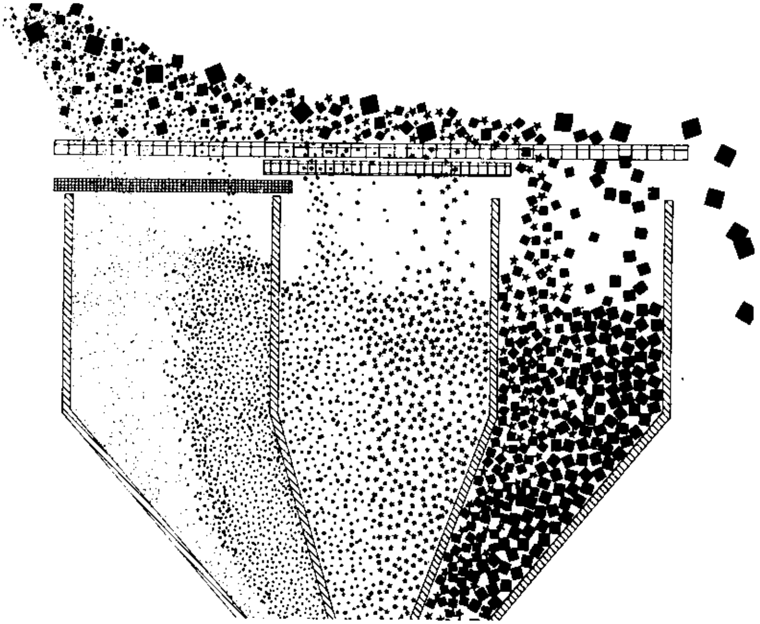


Figure 5.30 Segregation of Materials in the Hot Bins.

When material is drawn from the bin by opening a gate at the bottom, the stream consists predominantly of fine material at one edge and coarse material at the other. This condition is critical in the No. 1 (fine) bin, since the asphalt demand is influenced heavily by material from this bin. Therefore, the relative position of the sampling device (Figure 5.31) in the stream determines whether the sample will be composed of fine portion, coarse portion, or an accurate representation of the material in the bin (Figure 5.32).

Stratification of sizes in the fine bin may be caused by variation of grading in the stockpiles or by erratic feeding of the cold aggregate. When this form of segregation exists, representative samples cannot be obtained even when the sampling device is used correctly.

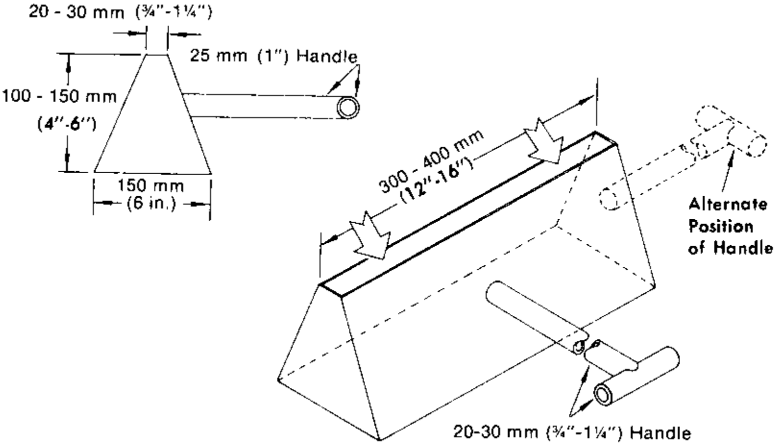


Figure 5.31 Asphalt Facility Aggregate Sampling Device.

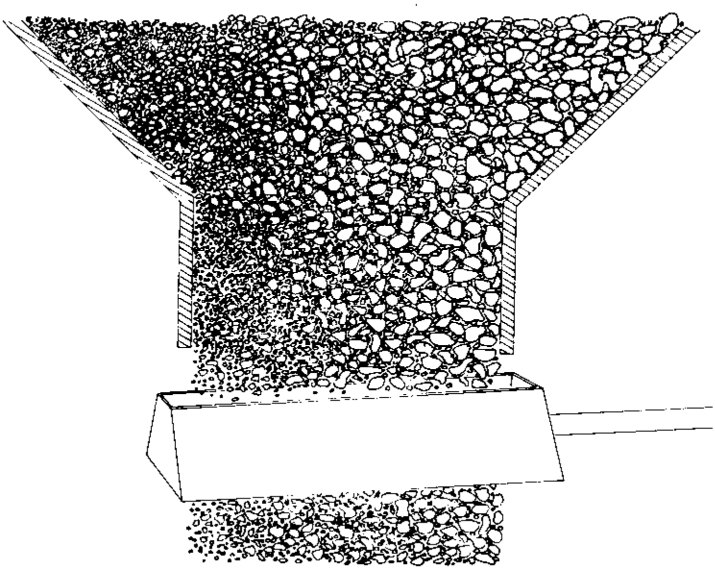


Figure 5.32 Correct Use of Sampling Device.

Aggregate Control at a Drum-Mixing Facility

Since the typical drum-mix facility, unlike a batch facility, does not incorporate a gradation screening unit, the aggregate must be proportioned prior to its entry into the mixing drum; this is essential. The most efficient way to accomplish this is with a multiple-bin cold-feed system equipped with precision belt feeders for the control of each aggregate. Under each bin is a belt feeder on to which the aggregate is proportioned. Precise controls (Figure 5.33) are used here to feed the exact proportions on to the belt. They must be calibrated to compensate for the moisture in the aggregate.

The plant should be equipped with provisions to conveniently obtain representative samples of the full flow of material from each cold feed and the total cold feed. Sieve analysis of dried aggregates are then performed as required by specifications.

Cold feed control consists of the following:

1. Sieve analysis of aggregate in each bin.
2. Calibrate feeders—both gate opening and belt speed.
3. Establish bin proportions.
4. Set gate openings and belt drive speeds.

Once calibrated, the gate openings should be checked frequently to ensure that they remain properly set. All settings should be considered tentative because the cold aggregate used in the mix may vary in grading and moisture content, which may require adjustments of the gates to maintain a uniform flow.

To calibrate the aggregate metering system and to plot a cold-feed capacity chart, a sampling device or method to obtain samples is necessary. The device should permit the flow of aggregate to be diverted to a collection container for accurate weight checks of timed aggregate samples (Figure 5.34). Such devices are usually installed at the end of the conveyor belt just prior to entry into the drum mixer.

Mineral Filler Control

When baghouse dust or commercial mineral filler is added to the mix, the amount used should be well controlled and checked frequently. One method is to determine the amount of filler used during the loading of several trucks. This and visual checking at frequent intervals should be done in addition to the calibration of the gate openings or weighing mechanisms. The gate opening is small and foreign matter, especially pieces of paper sacks in which the mineral filler is shipped, can obstruct the flow and sharply reduce the volume of filler added to the mixture. Since moisture can cause caking, which also obstructs the flow, measures should be taken to keep the filler dry. Calibration of the feeding and weighing mechanisms must be checked closely and frequently.

Temperatures

Determining Mixing Temperature

The mixing temperature for asphalt concrete is determined by the requirements for achieving dry aggregate, for good particle coating, and for laying and compacting. The aggregate must be sufficiently dry so that there is no foaming or slumping of the finished mixture or other evidence of moisture that may adversely affect its placement or service behavior. The

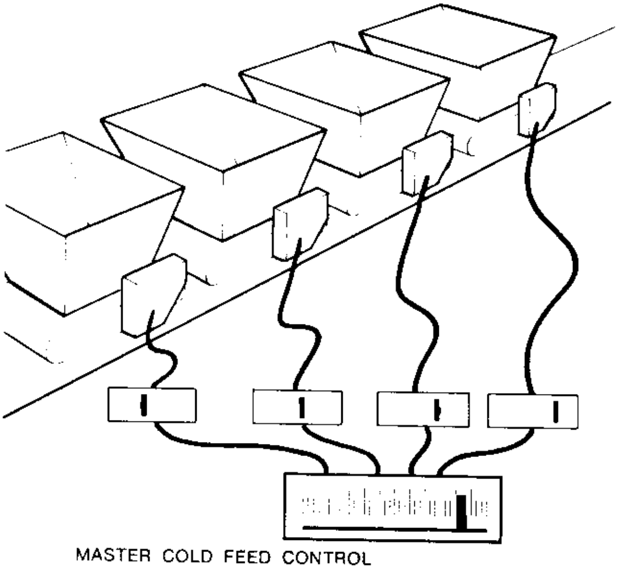


Figure 5.33 Master Cold Feed Control. *(Courtesy Barber-Greene Company)*

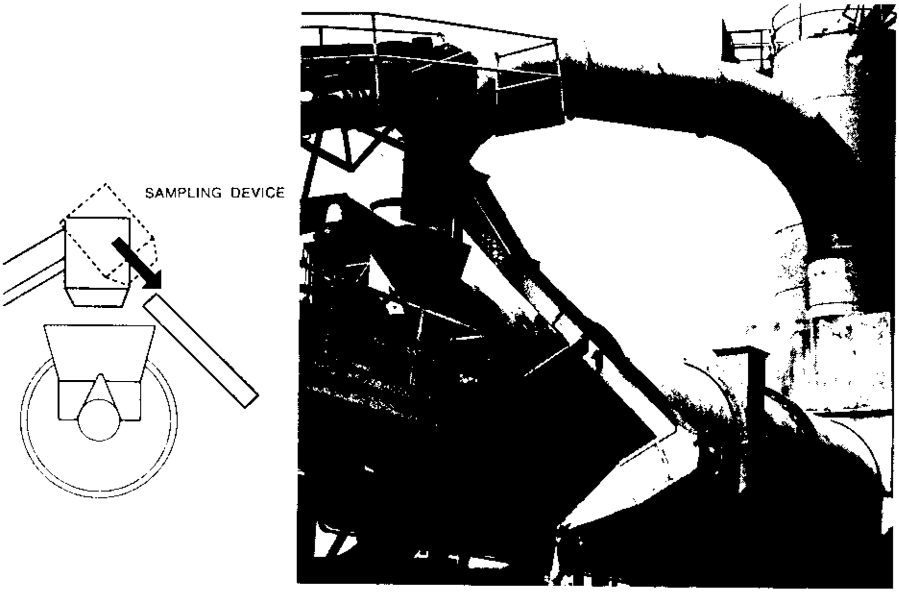


Figure 5.34 Typical Sampling Device.

lowest possible temperature at which these objectives can be attained should be used to reduce asphalt hardening and energy requirements. Therefore, temperatures for mixtures immediately after discharge from the pugmill may be as low as can be demonstrated to get good coating and compaction, but not more than 163°C (325°F).

Since the aggregate constitutes 90 to 95% of the mix, the temperature of the heated and dried aggregate controls the temperature of the asphalt-aggregate mixture in the pugmill. The temperature of the asphalt rapidly adjusts to that of the aggregate when the two are mixed.

Aggregate Temperatures

The temperature of the aggregate controls the temperature of the mixture, and the mixing temperature should be based on needs for placing and compacting. An aggregate temperature measuring device should be installed in the dryer discharge, in full view of the burner operator. This device is one of the most important plant control accessories and should be an accurate and reliable instrument.

The sensing element of the temperature indicator should have a shield sturdy enough to protect it from abrasion by the aggregate, but not so thick that it gives a distorted temperature reading. The sensing element should be installed to prevent excessive accumulated dust that may cause a time lag in measuring the temperature.

Temperature-measuring instruments should be checked frequently for accuracy. A simple way to do this is to submerge the sensing element along with an accurate thermometer in an oil bath. The oil is then heated and the readings of both devices are recorded as the oil approaches mixing temperatures.

Asphalt Temperature

To assure control of the asphalt temperature, an armored thermometer or pyrometer with a recorder should be placed in the asphalt feed line at a location near the discharge valve at the mixer unit. The asphalt storage tank should also be equipped with a recording thermometer having a minimum range of 24 hours. The temperature of the asphalt should be at or near the desired mixing temperature when it is added to the hot aggregate in the pugmill.

Batch Facility Operations

Basic Operations

Certain basic operations are common to all batch facilities.

- Aggregate storage and cold feeding.
- Aggregate drying and heating.
- Screening and storage of hot aggregates.
- Storage and heating of asphalt.

- Measuring and mixing of asphalt and aggregate.
- Loading of finished hot-mix.

Figure 5.35 illustrates the sequence of these operations.

Aggregates are removed from storage or stockpiles in controlled amounts and passed through a dryer where they are dried and heated. The aggregates then pass over a screening unit that separates the material into different sized fractions and deposits them into bins for hot storage. The aggregates and mineral filler (when used) are then withdrawn, in controlled amounts, combined with asphalt and thoroughly mixed in a batch. The mix is loaded into trucks and hauled to the paving site.

Proportioning Materials

Accurate proportioning of cold feed aggregates is extremely important. Except for whatever degradation may occur in drying, screening, and fines lost through the exhaust system, aggregate in hot bins is the same as in the cold feed. The only difference is that aggregates in the hot bins are separated into specific sizes.

If the material in the hot bin is recombined in the same proportions as it was deposited into each bin, the combined gradation will be the same as before the aggregate was separated. In order to determine the quantity of material to be withdrawn from each hot bin, a screen analysis is made of representative samples from each hot bin. The material from the bins is proportioned so that the combined gradation is that of the job-mix formula. Since cold feed materials were proportioned in a similar manner, the proportions of material leaving the hot bins should nearly equal the proportions of material deposited into each bin. Combining aggregates and setting feed gates were discussed in previous sections.

Aggregate Scales

A weigh hopper, completely enclosed by a dust shield, is suspended from a scale beam in such a way as to assure free movement. The lever system, knife edges, and bearings should be checked for cleanliness and to be sure that no moving part is binding against any other part. The dial needle should be free-swinging and register zero at no load.

The weigh hopper and scales should be checked with ten 25-kg, (or 50-lb) weights. With the screens running 250 kg (or 500 lb) of weights are attached to the weigh hopper and the exact dial recorded. The weights are removed and exactly 250 kg (or 500 lb) of aggregate are deposited in the weigh hopper. Check weights are again added, and the procedure repeated until the batch load is reached.

At each 250-kg (or 500-lb) increment, a test for sensitivity should be made by placing a 2.5 kg (5 lb) weight on the scale and checking for dial movement. Should the scales fail to conform to specification requirements, either in accuracy or sensitivity, plant production should not be permitted until necessary adjustments or replacements have been made.

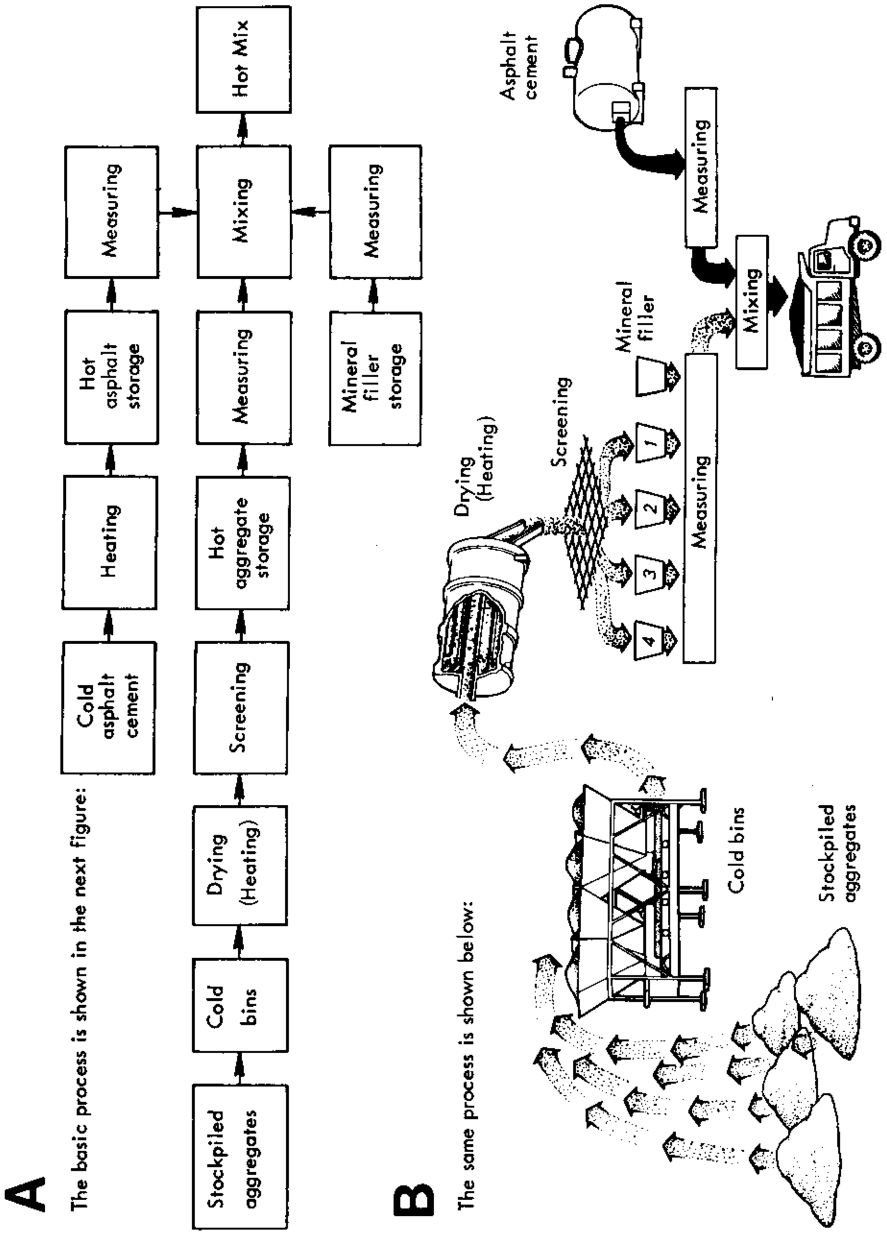


Figure 5.35 Basic Batch Facility Operations Shown (A) in Flow-Chart Form and (b) Schematically. *(Courtesy Tennessee Department of Transportation)*

Asphalt Scales

The asphalt scale is calibrated in nearly the same manner as the aggregate scale, but only one weighing operation is required. Standard weights are placed on or attached to the asphalt bucket; readings are recorded as each weight is added. This is continued until the combined weight is slightly in excess of the kilograms (pounds) of asphalt cement required per batch of paving mixture. Asphalt scales, if in true adjustment, should indicate the same value as the total of the standard weights used. If the weighing scale error exceeds the margin permitted by specifications, plant operations should not be started until the scales are adjusted or repaired by a qualified scale technician.

The tare weight of the empty asphalt bucket should be watched carefully to see that the bucket is drained completely and to compensate for any asphalt and dust clinging to it. The asphalt bucket should be tared at the beginning of each day and checked after the first few loads are discharged. Quite often, asphalt accumulates on the side and bottom of the bucket and solidifies overnight. This affects the tare weight and reduces the weight of asphalt actually used in the mix.

For an accurate zero reading, asphalt scales should not be checked until heating oil or steam is in the jacket around the asphalt bucket and the mixer.

Asphalt Meters

Asphalt meters are volume displacement mechanisms, and when used they should be checked for accuracy. Since asphalt content is usually expressed as a percent by weight, a correlation between meter readings and weight should be established.

A simple method to determine the correlation is to read the meter, pump a quantity of asphalt into a tared container, and then read the meter again. The weight of asphalt divided by the difference in meter readings determines the weight of asphalt pumped per division.

The viscosity and unit weight of the asphalt change with a change in temperature. When the temperature is increased the viscosity decreases. The unit weight decreases about 1 percent for an increase in temperature of 15°C (28° F). Pumping efficiency may be affected by a change in temperature, and it may be desirable to calibrate the pump over a range of asphalt temperatures. Volumes and viscosities can be determined later for calibration and plotting purposes if necessary.

Some asphalt meters have built-in temperature-compensating devices that correct the flow of asphalt when temperature changes occur. When a meter without a temperature-compensating device is used, it is necessary to adjust the delivery setting for each change in asphalt temperature.

Charging the Pugmill

Unless the batch facility is equipped to discharge all bins simultaneously, aggregate should be withdrawn from the hot bins into the weigh hopper, beginning with the largest size aggregate and progressing down to the finest size, with the material filler added last. This permits some mixing of the aggregate as it discharges into the mixer. The amount from each bin is weighed cumulatively on the scales. For example, suppose a 2000 kg (4409 lb) batch is mixed, using the scale settings determined as shown in Figure 5.36. The operator should carefully mark the scale reading for each bin withdrawal. The asphalt is weighed separately and should be introduced and distributed uniformly and quickly into the mixer. Otherwise, a nonuniform mixture could be produced.

Asphalt cut-off valves should be checked for positive action. They should close tightly so that no asphalt drips after the desired amount has been introduced or discharged from the weigh bucket.

Asphalt is introduced into the pugmill after the aggregate has been deposited from the weigh hopper. Wet mixing time begins when the asphalt is added. *Data:* Batch: 2000 kg (4409 lbs)

Asphalt Content: 5.5%

Aggregate: 2000 - 2000 (.055) = 1890 kg

(4409 - 4409 (.055) = 4166 lbs)

				Scale Reading	
Bin	**%**	**kg**	**lb**	**kg**	**(lbs)**
4	12	227	(500)	227	(500)
3	20	378	(833)	605	(1333)
2	28	529	(1167)	1134	(2500)
1	36	680	(1499)	1814	(3999)
MF	4	76	(167)	1890	(4166)
		1890	(4166)		

Figure 5.36 Sample Computation for Determining Scale Readings for Aggregate Weigh Hopper.

The Mixer

All mixer parts must be in good mechanical condition and in proper adjustment. Paddle faces can be set in a variety of combinations. The manufacturer's operation manual gives details on the correct settings of the paddle faces.

Clearance between the paddle tips and liner is governed by the maximum size aggregate and, normally, is less than half the maximum aggregate diameter. Dead spots may develop if the paddles have worn considerably or are broken and have not been readjusted or replaced.

Nonuniform mixing may occur if the mixer is overfilled (Figure 5.37). At maximum operating efficiency, the paddle tips should be barely visible in the material at the top of their periphery during mixing. Material above this level tends to float above the paddles and is not mixed. On the other hand, an insufficient batch will not be mixed properly either because there is not enough material to carry around in their paths (Figure 5.38). Both conditions can be minimized if the manufacturer's pugmill batch rating recommendation is followed. Normally, this is based on percentage of the capacity of the pugmill's "live zone" (Figure 5.39). It is the volume in cubic metres (cubic feet) below a line extending across the top arc to the inside walls of the pugmill. The volume of the shafts, liners, paddles, and tips are deducted.

Visual inspection of the mix and tests will indicate if the mixer is functioning satisfactorily and complete mixing is being obtained.

Mixing Time

During the mixing cycle asphalt, aggregates and mineral filler are blended in the pugmill. The length of time between opening of the weigh box hopper gate and the opening of the pugmill discharge gate is the batch mixing time. The batch mixing time must be long enough to produce a homogenous mixture of evenly distributed and uniformly coated aggregate particles. However, if the mixing time is too long, the lengthy exposure of the thin asphalt film to the high aggregate temperature in the presence of air can adversely affect the asphalt and reduce the durability of the mix.

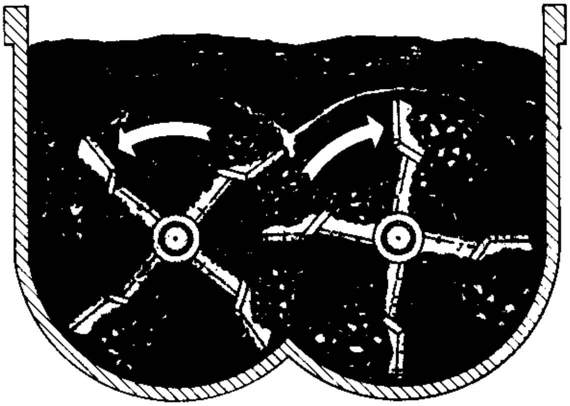


Figure 5.37 Overfilled Pugmill.

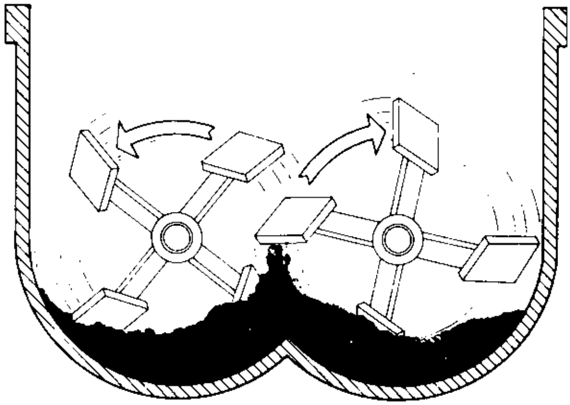


Figure 5.38 Underfilled Pugmill.

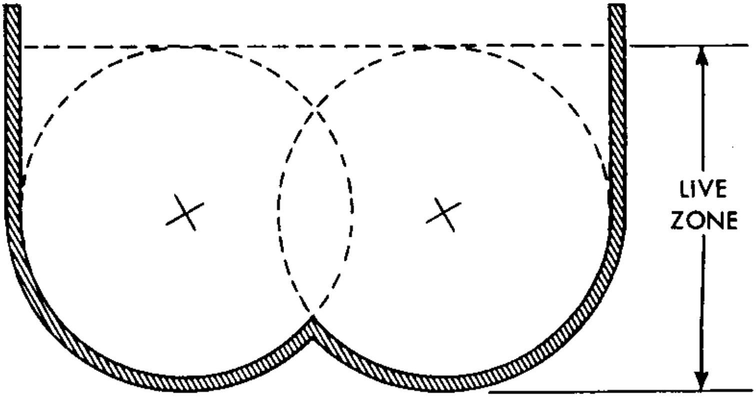


Figure 5.39 **Pugmill "Live Zone".**

Mixing time may be established for a particular mix in a facility by the procedure in AASHTO T 195, (ASTM D 2489). This method bases the degree of mixing on the percentage of coarse particles that are completely coated with asphalt. The method is correlated with mixing time.

The test involves separating coarse aggregate particles from the mix on a selected sieve size. About 200 to 300 particles should be examined under a strong light. Any speck of uncoated aggregate particle visible to the naked eye classifies that particle as uncoated. Usually, 90 and 95 percent coated particles are minimums for base and surface coarse mixes, respectively. The least time needed for the pugmill to achieve these minimums is the most desirable mixing time.

Drum-Mix Facility Operations

Basic Operations

The fundamental components of the drum-mix facility are:

- Aggregate cold-feed bins.
- Conveyor and aggregate weighing system.
- Drum mixer.
- Dust collection system.
- Hot-mix conveyor.
- Mix surge silo.
- Control van.
- Asphalt storage tank.

Referring to Figure 5.40, the following is a brief, general description of the sequence of processes involved in a typical drum-mix plant facility operation: Controlled gradations of aggregates are deposited in the cold feed bins (1) from which they are fed in exact proportions on to a cold-feed conveyor. (2) An automatic aggregate weighing system (3) monitors the amount of aggregate flowing into the drum mixer (4). The weighing system

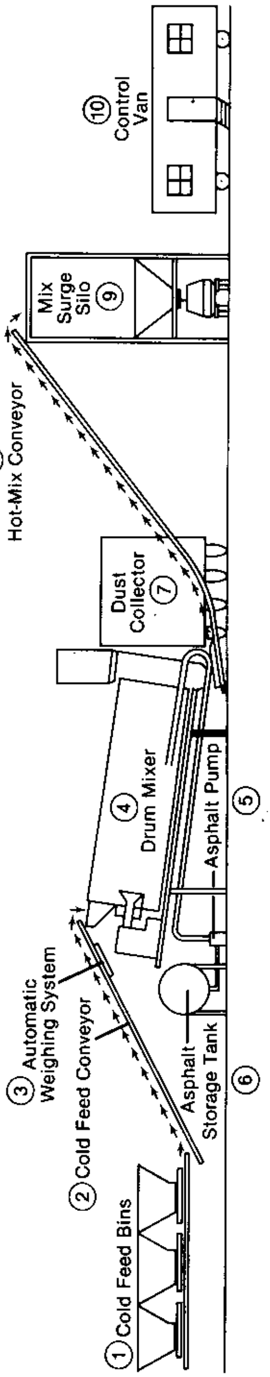


Figure 5.40 Basic Drum-Mix Facility.

is interlocked with the controls on the asphalt storage pump (5) which draws asphalt from a storage tank (6) and introduces it into the drum where asphalt and aggregate are thoroughly blended by the drum's rotating action. A dust collection system (7) captures excess dust escaping from the drum. From the drum, the hot-mix asphalt concrete is transported by hot-mix conveyor (8) to a surge silo (9) from which it is loaded into trucks and hauled to the paving site. All plant operations are monitored and controlled from instruments in the control van (10).

The mixing process is essentially the same in all drum-mix facilities. Methods of feeding the materials to the drum mixer, however, may differ.

The production of hot-mix that meets job specification is most easily accomplished when the various parts and functions of the plant are in balance; that is, when they are properly coordinated to work together as a smooth-working unit. Also essential for consistent and high quality hot-mix asphalt is uniform (uninterrupted) operation. Accurate proportioning of materials is entirely dependent on the uniform flow of those materials. Stops and starts adversely affect mix quality.

Aggregate Control

In a drum-mix facility, mix gradation and uniformity are entirely dependent on the cold-feed system. Proper care must be exercised not only in production of the aggregate, but also in storage and handling. The contractor should provide for receiving and handling aggregates in such a way that there is no danger of contamination or intermingling. Among other things, this means providing clean surfaces on which to place the materials.

Stockpiles must be properly graded, and it is recommended that stockpiles be split into different-sized fractions to properly control the gradation of the mix.

Since the typical drum-mix facility, unlike a batch facility, does not incorporate a gradation screening unit, the aggregate must be proportioned prior to its entry into the mixing drum; this is essential. The most efficient way to accomplish this is with a multiple-bin cold-feed system equipped with precision belt feeders for the control of each aggregate. Under each bin is a belt feeder onto which the aggregate is proportioned. Precise controls are used here to feed the exact proportions onto the belt. As the aggregate is fed into the drum by the conveyor belt, it passes over the weigh bridge. The weigh bridge is a belt idler, or supporting roller, mounted on a scale (Figure 5.41). As the aggregate passes over the idler its weight is sensed, and transmitted as a visual display on the control console. The speed of the conveyor belt is also taken into consideration, so that the visual display indicates weight of aggregate per hour. This value is the basis of the aggregate-asphalt blending system.

The weight of aggregate per hour going into the drum is the basis of the total mix formulation. Since the weight of the aggregate includes moisture, the weight of the moisture must be subtracted to arrive at the true aggregate feed rate. The actual moisture content is periodically determined by moisture extraction tests.

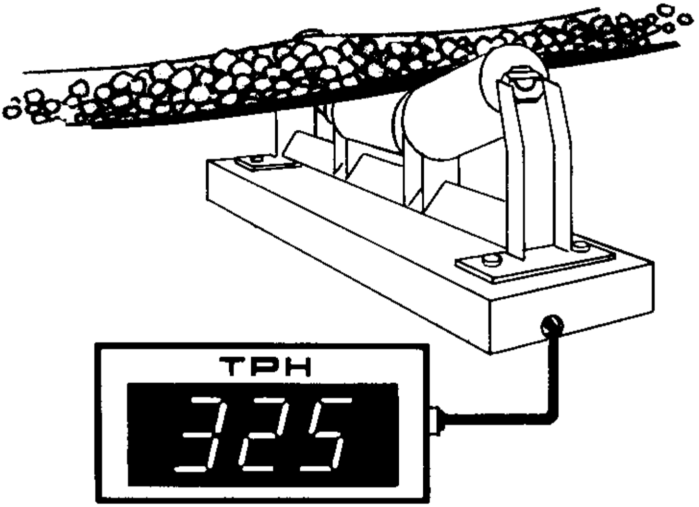


Figure 5.41 Weigh Bridge.

Asphalt Metering

The drum-mixer is typically equipped with a device (Figure 5.42) to add asphalt to the aggregate inside the drum mixer.

The asphalt metering and delivery system is a continuous mechanical proportioning system interlocked with the aggregate weigh system to ensure the exact asphalt content of the mix. The weight of aggregate going into the mixer, as measured by the weigh belt, is the basis of determining the quantity of asphalt delivered into the drum.

The proportioning of asphalt is accomplished by establishing the necessary rate of asphalt delivery in litres (gallons) per minute to match the aggregate delivery in tonnes (tons) of dry aggregate per hour. The asphalt cement delivery rate is increased or decreased proportionately according to the corrected dry weight measurement of aggregate passing over the belt scale. The rate of asphalt delivery is indicated on a rate meter on the control panel.

Typically, the rates of delivery of aggregate and asphalt cement are recorded on continuously recording circular graphs located in the control van. The graphs provide both monitoring and a permanent record of the proportioning of asphalt cement and total aggregate.

Mixing

The drying and blending of the aggregate, and the mixing of the asphalt and aggregate takes place in the drum. The drum-mixer can be divided into two sections or zones: (1) a primary or radiation zone and (2) a secondary or convection/coating zone (Figure 5.43).

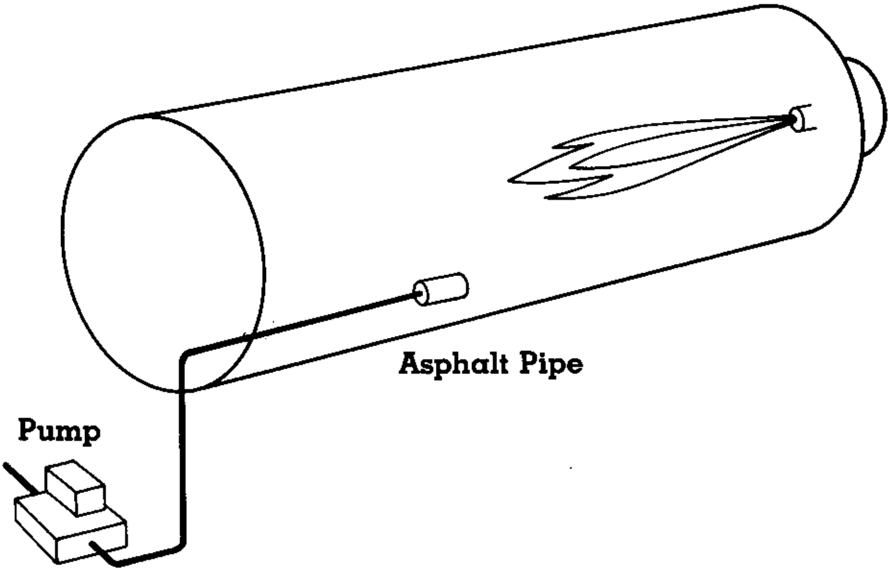


Figure 5.42 Asphalt Inlet.

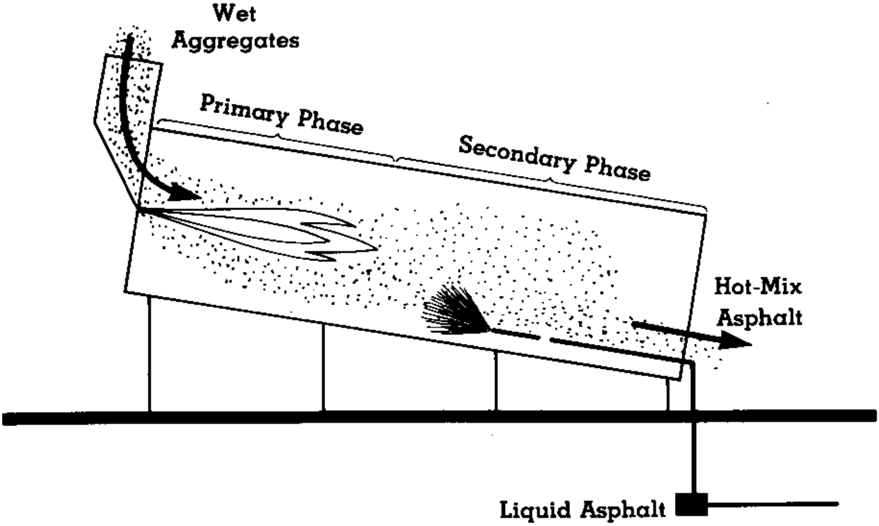


Figure 5.43 Zones in Drum Mixer.

Aggregates enter the primary zone, where heat from the burner dries and heats them. They then move to the secondary zone, where asphalt is added and aggregates and asphalt are thoroughly blended. Continued convection drying also occurs in the secondary zone. The mixture of hot asphalt and moisture released from the aggregate produces a foaming mass that traps the fine material (dust) and aids in the coating of the larger particles.

Surge and Weigh Scale

In a drum-mix operation, which produces a continuous flow of fresh asphalt hot mix, it is necessary to have a surge silo for temporary storage of the material and for controlled loading of trucks. A weigh system may be connected to the holding bin of the silo to monitor the amount of material loaded into each truck. Weight measurements are normally recorded by the weigh system control panel, located in the control van or trailer.

Miscellaneous Inspection Duties

Inspection of Trucks

Truck bodies should not leak or have deep indentations that might cause the asphalt mix to stick. The truck bed should also be free of material that may be detrimental to the mix. After being cleaned the inside surface of truck bodies should be treated with a thin coating of a non-petroleum based release agent to prevent material from sticking. The excess solution must be drained off before asphalt mix is placed in the truck.

The bodies of the truck beds should be insulated and covered if necessary to maintain the mix temperature within specification requirements.

Observing the Mix

Finished mix should be frequently observed. No testing method or device is as quick or convenient as the human eye.

Temperature control is stressed in all phases of hot-mix asphalt production. This is a primary factor in controlling quality. The appearance of the mix in the correct temperature range should be recognized by visual inspection. The emission of blue smoke from the mix in the truck is often an indication of overheating. If the mix is underheated it may appear stiff or sluggish as it is deposited in the truck and may have a nonuniform distribution of asphalt. An abnormally peaked condition of the pile in the truck may also indicate underheating.

If the temperature does not appear to be correct, it should be measured with a thermometer (Figure 5.44).

A stiff mix may also indicate an asphalt deficiency, particularly if it also has a lean, dull appearance. Mix slumped in the truck may indicate the mix contains too much asphalt or excessive moisture.

Personnel should become familiar with the appearance and physical characteristics of the mix used on the project. Trial mixtures prepared in the central or field laboratory should be observed closely. Table 5.3 gives probable causes and corrective measures for certain observed conditions.

Table 5.3 Possible Causes of Mix Deficiencies in Hot-Mix Asphalt.

Aggregates Too Wet	Inadequate Bunker Separation	Aggregate Feed Gates Not Properly Set	Over-Rated Dryer Capacity	Dryer Set Too Steep	Improper Dryer Operation	Temp. Indicator Out of Adjustment	Aggregate Temperature Too High	Worn Out Screens	Faulty Screen Operation	Bin Overflows Not Functioning	Leaky Bins	Segregation of Aggregates in Bins	Carryover in Bins Due to Overloading Screens	Aggregate Scales Out of Adjustment	Improper Weighing	Feed of Mineral Filler Not Uniform	Insufficient Aggregates in Hot Bins	Improper Weighing Sequence	Insufficient Asphalt	Too Much Asphalt	Faulty Distribution of Asphalt to Aggregates	Asphalt Scales Out of Adjustment	Asphalt Meter Out of Adjustment	Undersize or Oversize Batch	Mixing Time Not Proper	Improperly Set or Worn Paddles	Faulty Dump Gate	Asphalt and Aggregate Feed Not Synchronized	Occasional Dust Shakedown in Bins	Irregular Plant Operation	Faulty Sampling	*Types of Deficiencies That May Be Encountered in Producing Plant-Mix Paving Mixtures.*
		A												B	B				A	A	A	B	C	B	B	B		C			A	Asphalt Content Does Not Check Job Mix Formula
	A	A						B	B	B	B	A	A	B	B	B	A							B		B	B	C	B		A	Aggregate Gradation Does Not Check Job Mix Formula
	A	A							B	B	B	A	A	B	B	B	A							B	B			C	B		A	Excessive Fines in Mix
A			A	A	A	A	A																							A		Uniform Temperatures Difficult to Maintain
											B			B	B									B								Truck Weights Do Not Check Batch Weights
														B	B					A	A	B	C	B		B		C				Free Asphalt on Mix in Truck
																		B									B					Free Dust on Mix in Truck
A			A	A	A	A													A		A	B	C	B	B	B		C		A		Large Aggregate Uncoated
									B	B	A	A	A	B	B	B	A	B			A	B	C		B	B	B	C	B	A		Mixture in Truck Not Uniform
																		B			A			B	B	B				A		Mixture in Truck Fat on One Side
					A															A	A	B	C	B				C		A		Mixture Flattens in Truck
		A			A	A	A																							A		Mixture Burned
A			A	A	A	A			B										A			B	C	B				C		A		Mixture Too Brown or Gray
														B	B	B	A			A	A	B	C	B				C		A		Mixture Too Fat
					A	A	A																							A		Mixture Smokes in Truck
A			A	A	A	A																								A		Mixture Steams in Truck
					A	A	A												A										A	A		Mixture Appears Dull in Truck

A - applies to batch and drum-mix facilities.
B - applies to batch facilities.
C - applies to drum-mix plant facilities.

Sampling and Testing

Purpose of Tests

Sampling and testing are two of the most important functions in facility control. Data from these tests are the tools with which the quality of the product is controlled. For this reason, great care must be used in following correct sampling and testing procedures. Sampling is especially important.

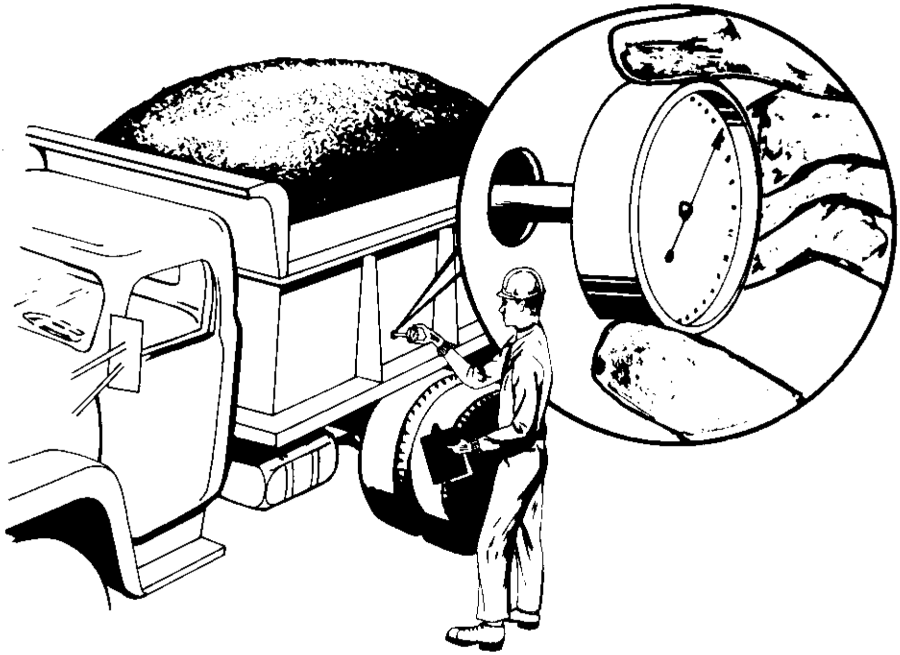


Figure 5.44 Measuring Temperature of Mix in Truck.

One of the greatest single sources of error in materials testing is the failure to obtain a representative sample.

Accurate sieve analyses are basic to good hot-mix design and control. Since time is an important factor, dry sieve analyses are made whenever possible. The washed sieve analysis of the total aggregate gives more accurate results, and it should be made frequently as a check on the dry sieve analysis.

An appreciable change in gradation will be reflected in a change in some other characteristics of the mix, such as a deficiency or an excess of asphalt. The mix will change in appearance and take on a different surface texture when compacted.

Field laboratory compacted specimens are useful for control of compaction by paving forces as well as for checking the properties of compacted mix.

Extraction tests measure the asphalt content and provide aggregate for the gradation analysis. This is the final check on all individual operations that have gone into producing the mixture.

Sampling Schedule

In controlling operations, it is necessary to obtain numerous samples at various points along the production line. On projects that have no specified schedule or method of sampling, Table 5.4 may be used as a guide. More frequent testing is usually necessary at the start of a project and when production becomes erratic.

Table 5.4 Suggested Schedule of Sampling and Testing

Sample of	(1) Minimum Sample Frequency	(2) Minimum Size of Sample	Test to be Performed	Test Method Designation
Cold Aggregate	(1) As Necessary	See Table 5.5	Sieve Analysis Specific Gravity Sand Equivalent (When Required)	AASHTO T-11 AASHTO T-27 AASHTO T-84 (ASTM C128) AASHTO T-85 AASHTO T-176
Each Hot-Bin	2 Daily	(3) See Table 5.5	Sieve Analysis Sand Equivalent (When Required)	AASHTO T-11 AASHTO T-27 AASHTO T-176
Hot Aggregate, Combined	2 Daily	See Table 5.5	Sieve Analysis	AASHTO T-11 AASHTO T-27
Mineral Filler	(1) As Necessary	See Table 5.5	Sieve Analysis	AASHTO T-37
Asphalt	2 for each transport delivered (4)	1 litre (1 Quart)	Send to Central Laboratory	AASHTO T-40
Uncompacted Mix	2 daily	9 kg (20 lb.)	Complete Extraction	AASHTO T-168 (ASTM D 979) AASHTO T-164 (ASTM D 2172) AASHTO T-30
Uncompacted Mix	2 Daily	7 kg (15 lb.)	Density Stability	AASHTO T-209 (ASTM D 2041) Project Specification Requirements

NOTES:

(1) Sample frequency will be governed by instructions of the contracting agency and by the immediate conditions surrounding the given project.

(2) Sample size may be governed by instructions of the contracting agency. For special conditions sample size may vary.

(3) In batch plants, each hot-bin sample should be taken from the flow of material from the bin gate in increments from several successive batches, and in continuous mix plants in several increments spaced several seconds apart.

(4) One sample for check testing and one sample to hold in reverse if question arises.

Field Mix Design

Regardless of the design method being used, the overall objective is to determine an economical blend and gradation of aggregates mixed with asphalt within the limits of the specifications. The contractor is normally furnished a mix design that has been developed on the basis of preliminary investigation. This is the guide to which he must fit his job-mix formula. While the two may not be identical, the contractor must develop the job-mix formula to conform as nearly as possible to the design furnished him.

Table 5.5 Size of Samples

Nominal Maximum Size of Particles, Passing Sieve		Minimum Weight of Field Samples	
Standard	Alternate	kg	lb
	Fine Aggregate		
2.36 mm	No. 8	5	10
4.75 mm	No. 4	5	10
	Coarse Aggregate		
9.5 mm	⅜ in.	5	10
12.5 mm	½ in.	10	20
19.0 mm	¾ in.	15	30
25.0 mm	1 in.	25	50
37.5 mm	1½ in.	30	70
50.0 mm	2 in.	40	90
63 mm	2½ in.	45	100
75 mm	3 in.	60	125
90 mm	3½ in.	65	150

Records and Reports

Adequate records must be kept by the facility inspector. They furnish the basis on which compliance with specifications is determined and on which payments are made. They should, therefore, be clear, complete, and accurate. Records also provide a history of the construction and the materials that went into the project. They furnish a basis for all future studies and evaluations of the project.

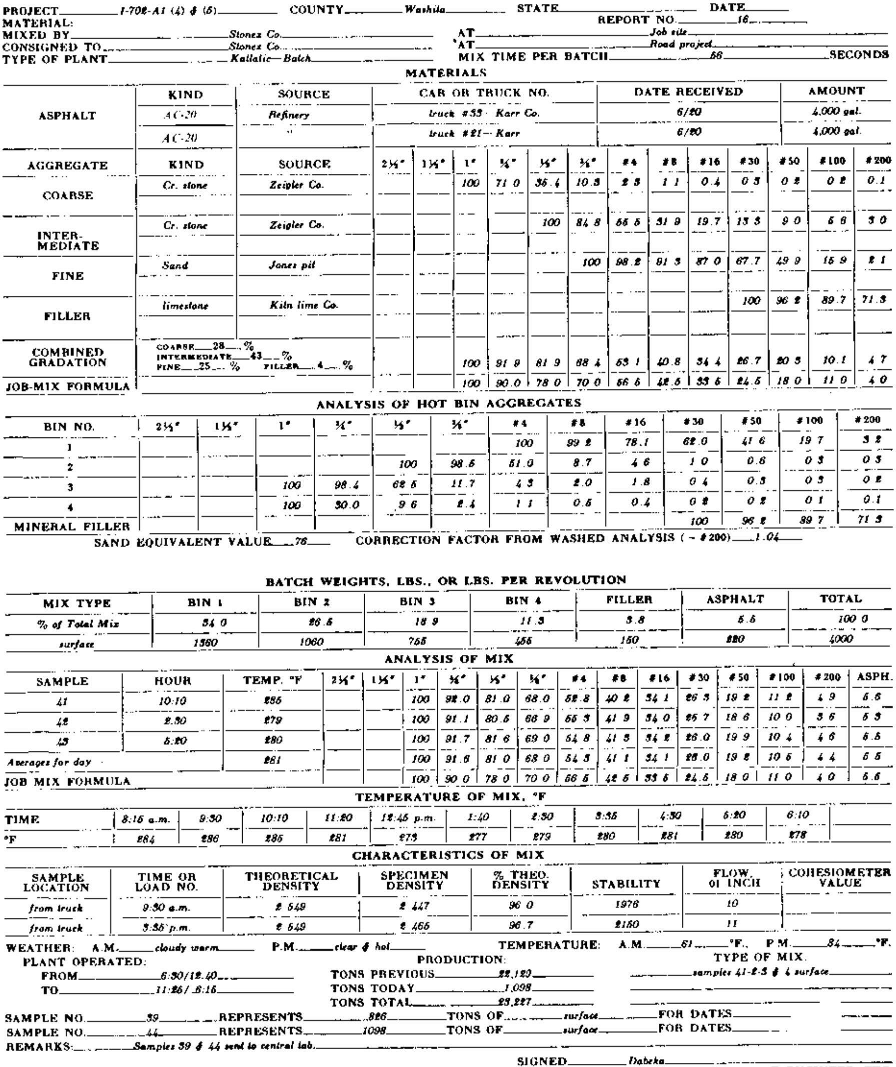
The plant inspector should keep a diary of every project. It should be kept in such a manner that duplicate copies may be made and distributed. The initial entry should contain such standard information as:

1. Project number,
2. Facility location,
3. Type and make of facility,
4. Source of materials,
5. Names of key personnel, and
6. Other pertinent data.

In addition to dates and routine weather comments, the inspector should record the principal facility activities, inspection personnel; and briefly describe the daily operations. Unusual events should be noted, particularly those which may affect the paving mix.

The plant inspector should keep a daily summary report of plant activities. On the heading of this form, the same information should be recorded as shown on the daily diary report. There should be a summary of the results of all tests performed during the day and a tabulation of the amounts of material received and used. A specimen daily report is shown in Figure 5.45.

A field record should also be kept on the locations in which the asphalt mix is placed in the roadway by lanes, courses, and stations. This information is listed on separate forms.



PROJECT *I-702-A1 (4) & (6)* COUNTY *Washila* STATE ____ DATE ____
REPORT NO. *16*
MATERIAL:
MIXED BY *Stonex Co.* AT *Job site*
CONSIGNED TO *Stonex Co.* AT *Road project*
TYPE OF PLANT *Kallatic—Batch* MIX TIME PER BATCH *55* SECONDS

MATERIALS

	KIND	SOURCE	CAR OR TRUCK NO.	DATE RECEIVED	AMOUNT
ASPHALT	*AC-20*	*Refinery*	*truck #33 · Karr Co.*	*6/20*	*4,000 gal.*
	AC-20	*"*	*truck #21—Karr*	*6/20*	*4,000 gal.*

AGGREGATE	KIND	SOURCE	2½"	1½"	1"	¾"	½"	⅜"	#4	#8	#16	#30	#50	#100	#200
COARSE	*Cr. stone*	*Zeipler Co.*			100	71 0	36.4	10.3	2 3	1 1	0.4	0 3	0 2	0 2	0.1
INTER-MEDIATE	*Cr. stone*	*Zeigler Co.*					100	84 8	55 5	31 9	19.7	13 3	9 0	5 6	3 0
FINE	*Sand*	*Jones pit*						100	98.2	81 3	87 0	67.7	49 9	15 9	2 1
FILLER	*limestone*	*Kiln lime Co.*										100	96 2	89.7	71.3
COMBINED GRADATION	COARSE *28* % INTERMEDIATE *43* % FINE *25* % FILLER *4* %				100	91 9	81 9	68 4	53 1	40.8	34 4	26.7	20 3	10.1	4 7
JOB-MIX FORMULA					100	90.0	78 0	70 0	56 5	42.5	33 5	24.5	18 0	11 0	4 0

ANALYSIS OF HOT BIN AGGREGATES

BIN NO.	2½"	1½"	1"	¾"	½"	⅜"	#4	#8	#16	#30	#50	#100	#200
1							100	99 2	78.1	62.0	41 6	19 7	3 2
2					100	98.5	51.0	8.7	4 6	1 0	0.6	0 3	0 3
3			100	98.4	62 5	11.7	4 3	2.0	1.8	0 4	0.3	0 3	0 2
4			100	30.0	9 6	2.4	1 1	0.5	0.4	0 2	0 2	0 1	0.1
MINERAL FILLER										100	96 2	89 7	71 3

SAND EQUIVALENT VALUE *76* CORRECTION FACTOR FROM WASHED ANALYSIS (− #200) *1.04*

BATCH WEIGHTS, LBS., OR LBS. PER REVOLUTION

MIX TYPE	BIN 1	BIN 2	BIN 3	BIN 4	FILLER	ASPHALT	TOTAL
% of Total Mix	54 0	26.5	18 9	11.3	3.8	5.5	100 0
surface	1560	1060	755	455	150	220	4000

ANALYSIS OF MIX

SAMPLE	HOUR	TEMP. °F	2½"	1½"	1"	¾"	½"	⅜"	#4	#8	#16	#30	#50	#100	#200	ASPH.
41	10:10	285			100	92.0	81.0	68.0	52.8	40 2	34 1	26 3	19 2	11 2	4 9	5.6
42	2:30	279			100	91.1	80.5	66 9	55 3	41 9	34 0	25 7	18 6	10 0	3 6	5 3
43	5:20	280			100	91.7	81 6	69 0	54 8	41 3	34 2	26.0	19 9	10 4	4 6	5.5
Averages for day		281			100	91.6	81 0	68 0	54 3	41 1	34 1	26.0	19 2	10 5	4 4	5 5
JOB MIX FORMULA					100	90 0	78 0	70 0	56 5	42 5	33 5	24.5	18 0	11 0	4 0	5.6

TEMPERATURE OF MIX, °F

TIME	8:15 a.m.	9:30	10:10	11:20	12:45 p.m.	1:40	2:30	3:35	4:30	5:20	6:10	
°F	284	286	285	281	273	277	279	280	281	280	278	

CHARACTERISTICS OF MIX

SAMPLE LOCATION	TIME OR LOAD NO.	THEORETICAL DENSITY	SPECIMEN DENSITY	% THEO. DENSITY	STABILITY	FLOW, 01 INCH	COHESIOMETER VALUE
from truck	9:30 a.m.	2 549	2 447	96 0	1976	10	
from truck	3:35 p.m.	2 549	2 466	96.7	2150	11	

WEATHER: A.M. *cloudy warm* P.M. *clear & hot* TEMPERATURE: A.M. *61* °F., P.M. *84* °F.
PLANT OPERATED: PRODUCTION: TYPE OF MIX.
FROM *6:30/12:40* TONS PREVIOUS *22,129* *samples 41-2-3 & 4 surface*
TO *11:25/ 5:15* TONS TODAY *1,098*
TONS TOTAL *23,227*
SAMPLE NO. *39* REPRESENTS *826* TONS OF *surface* FOR DATES ____
SAMPLE NO. *44* REPRESENTS *1098* TONS OF *surface* FOR DATES ____
REMARKS: *Samples 39 & 44 sent to central lab.*
SIGNED *Dabeka*
PLANT INSPECTOR, RESIDENT ENGINEER, ETC.

(Note This report is intended as a guide. When more materials are being used, or more than one mix type is being produced, additional spaces may be necessary. Metric equivalents are omitted because U.S. plants are manufactured to customary units only.)

Figure 5.45 Asphalt Plant Inspector's Daily Report.

Chapter 6
Hot-Mix Asphalt Paving

It is not feasible to examine in any detail all the different hot-mix asphalt paving machines in use today. Rather, this chapter describes paving fundamentals. This discussion provides the reader with enough background to adapt to a given paving machine design.

In addition to describing asphalt mix pavers and the actual placement procedure, this chapter contains information on various other items of equipment and tools used in the process.

Finally, information is presented on the necessary measures needed to prepare the new and existing roadway surface prior to paving.

SECTION 6.1 Asphalt Paving Equipment

☐ Introduction
☐ The Asphalt Paver
☐ Auxiliary Equipment

SECTION 6.2 Roadway Surface Preparation

☐ Introduction
☐ Preparation of Unpaved Surfaces
☐ *Preparation of Old Pavement Surfaces*
☐ Leveling Courses

SECTION 6.3 Placing Hot-Mix Asphalt

☐ Introduction
☐ Receiving the Mix
☐ Placing with the Paver
☐ Constructing Paving Joints

SECTION 6.4 Supplemental Roadway Paving and Appurtenances

☐ Introduction
☐ Roadway Widening
☐ Shoulders
☐ Superelevation
☐ Asphalt Curbs and Dikes
☐ Asphalt Ditches and Spillways
☐ Asphalt Slopes and Revetments

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2. *Principles of Construction of Hot-Mix Asphalt Pavements, MS-22,* Asphalt Institute.

VISUAL AIDS

1. *Placing Asphalt Hot Mix, VA-14,* slide show (20 min.), Asphalt Institute.

6.1 Hot-Mix Asphalt Paving Equipment

Introduction

The Paving Operation

Spreading and compacting asphalt and aggregate mixtures is the operation to which all the other processes are directed. Aggregates have been selected and combined; the mix designed; the mixing facility and its auxiliary equipment set up, calibrated and inspected; and the material mixed together and delivered to the paver (Figure 6.1).

Asphalt mix is brought to the paving site in trucks and deposited directly into the paver or in some cases windrows in front of the paver. The paver then spreads the mix at a set width and thickness as it moves forward. In doing so, the paver partially compacts the material. Immediately or shortly thereafter and while the mix is still hot, steel-wheeled, vibratory or rubber-tired rollers are driven over the freshly-paved strip, further compacting the mix. Rolling is usually continued until the pavement is compacted to the required density, or the temperature has dropped to a point where further compaction may produce detrimental results (see Chapter 7).

After the pavement course has been compacted and allowed to cool, it is ready for additional paving courses or ready to support traffic loads.

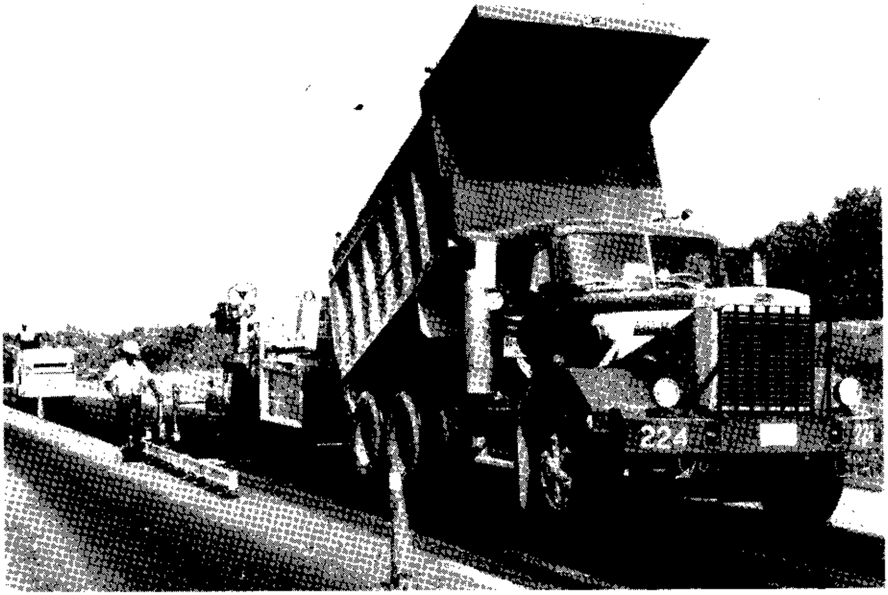


Figure 6.1 Hot-Mix Asphalt Being Delivered to Paver.

Planning and Preparation

Paving operations require careful planning and preparation. The surface to be paved must be properly prepared. Enough vehicles and equipment must be available and in good operation to provide a steady flow of materials and progress without delays. Plant production must be closely coordinated with the paving operation, and the compaction of freshly placed mixture must be prompt and adequate.

Asphalt Paving Equipment

Most asphalt-aggregate mixtures produced at a mixing facility are placed by asphalt pavers. This is the basic item of paving equipment. Other equipment used in connection with the paving operation includes: the asphalt distributor, motor grader, windrowing equipment, trucks, hand tools, and other machinery and implements.

The Asphalt Paver

Operation of the Paver

The asphalt paver spreads the mixture in a uniform layer of desired thickness and shape, or finishes the layer to the desired elevation and cross section, ready for compaction. Modern pavers are supported on crawler treads or pneumatic-tired wheels. These machines can place a layer of less than 25 mm (1 in.) to approximately 250 mm (10 in.) in thickness over a width of 1.8 to 9.8 m (6 to 32 ft.) Working speeds generally range from 3 to 20 m (10 to 70 ft) per minute, (Figure 6.2).

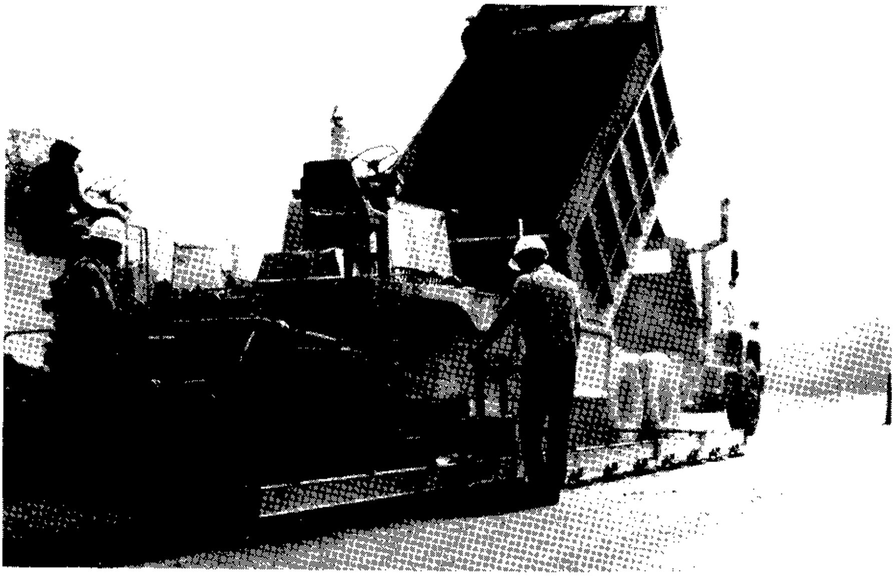


Figure 6.2 Asphalt Paver.

The paver is a relatively large machine with many intricate parts and adjustments. Pavers in use today may differ in detail, but they are similar in principle.

The plan and side views shown in Figure 6.3 trace the flow of asphalt mix from the receiving hopper at the front of the paver to the finished pavement behind the screed at the rear of the machine. The mix is dumped into the receiving hopper at the front of the machine from a truck that is pushed ahead by the paver. Rollers mounted on the front of the paver contact the rear tires of the truck and allow the finisher to push the truck while it is dumping into the hopper.

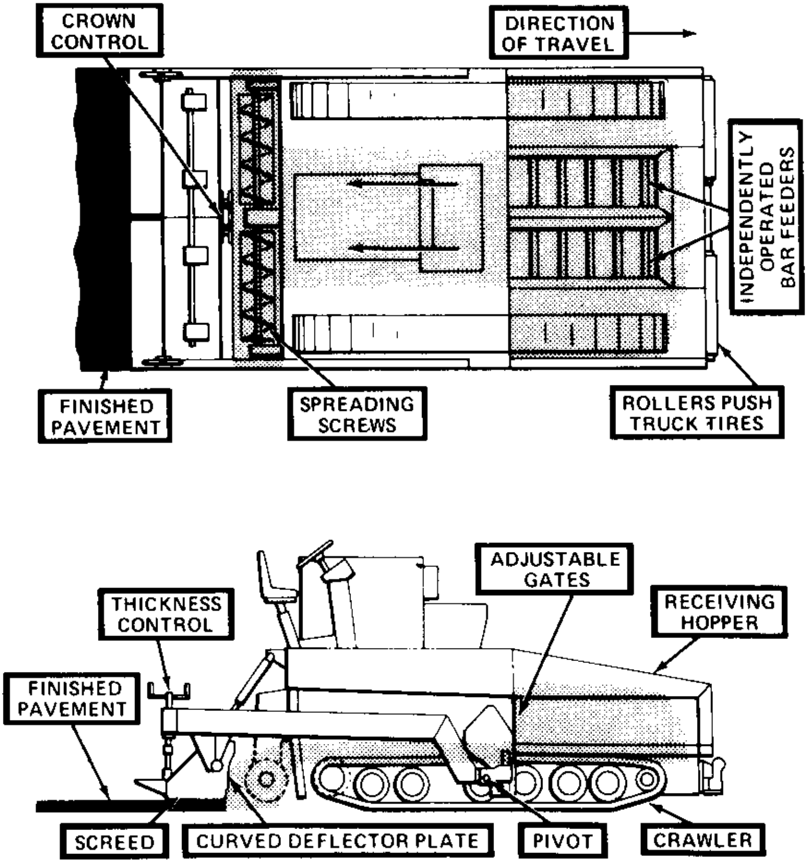


Figure 6.3 Flow of Material Through the Asphalt Paver.

Another method can be used for charging the asphalt paving machine with hot mix. Mix placed ahead of the paver in a calibrated windrow is picked up by an attachment mounted on the paver. This unit is referred to as a low-lift loader (Figure 6.4) and it picks up the material and deposits it in the receiving hopper as the paver moves forward.

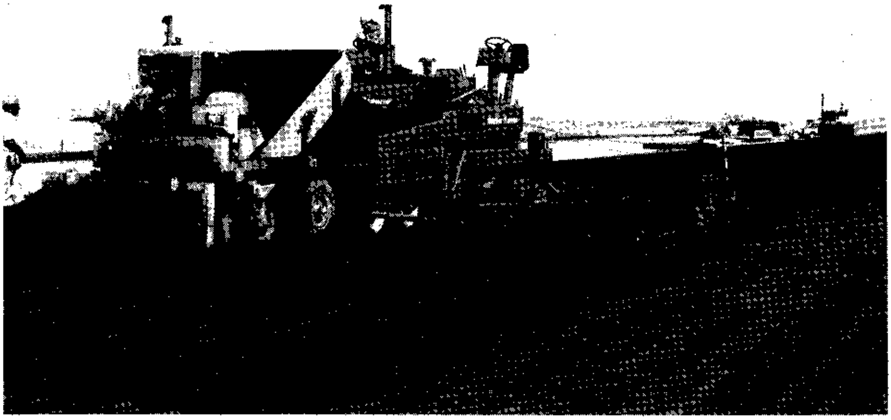


Figure 6.4 Low Lift Loader Being Used to Transfer Asphalt Mix From Windrow to Paver Hopper.

After receiving the material in the hopper, two independently controlled slat feeders carry the mix back through the control gates to the spreading screws. Each spreading screw and its respective feeder are synchronized, permitting the operator to accurately distribute the mix in front of the screed unit.

The screed unit is attached to the tractor unit by two long pull arms that pivot well on the paver (Figure 6.5). The arms provide no support for the screed when it is in an operating position. As the tractor pulls the screed into the material, the screed will seek the level where the path of its bottom surface is parallel to the direction of the operating position. As the tractor pulls the screed into the material, the screed seeks the level where the path of its bottom surface is parallel to the direction of the pull.

Cut-off shoes as well as extensions to the screed unit may be installed to limit the width of the mat or to extend the width beyond the normal width of the screed unit.

There are several different types of extensions for a paver to widen the width of material being placed. A distinction however should be made as to whether the screed is actually being extended or a thick metal plate is simply being extended which is not really a screed extension but is more properly called a strikeoff. The distinction is that in the case of using the hydraulic strikeoff, the mix is simply being cast out to the side of a normal width screed and is being struck off by a narrow steel plate. The strikeoff does not have auger extensions nor does it have the capacity to vibrate or tamp the mix. As a result, with the use of this unit, surface texture and density may vary. Whether or not texture, thickness, and density remain unaffected after compaction behind a screed or after strikeoff is greatly dependent upon the skill of the paving operator. Consequently, many agencies do not permit the use of paver extensions on surface courses.

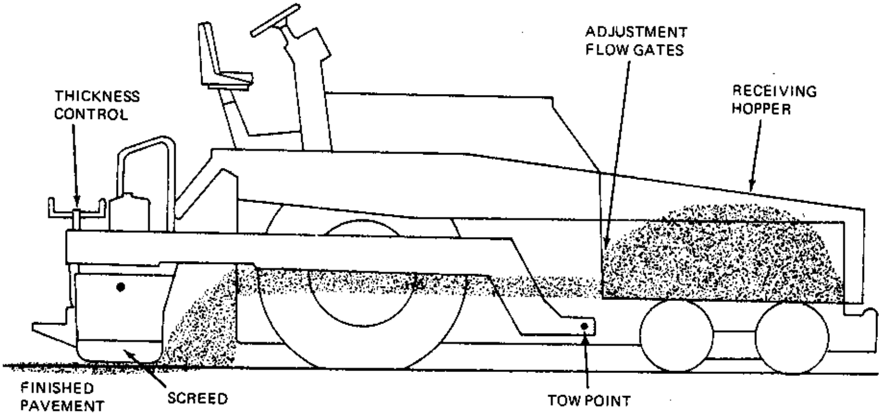


Figure 6.5 **Flow of Material Through the Self-Propelled Asphalt Paver.** *(Courtesy Barber-Greene Company)*

When difficulty arose with non-uniform conditions using extensions, the paving equipment industry designed new fully extendable screeds that also extended hydraulically in front of or in back of the main screed. These new extensions of variable width placement generally have full vibration and for a fixed width (for long stretches of wide paving) can be equipped with auger extensions as well. Persons responsible for paving operations should acquaint themselves with the various features of the different manufacturers' equipment to ensure that the mat thickness, texture, and density is as uniform as possible.

The Tractor Unit

The asphalt paver consists essentially of a tractor unit and a screed unit. The tractor unit provides the motive power through crawlers or pneumatic tires traveling on the road base. It includes the power plant, receiving hopper, feed conveyor, spreading screws, controls, and operator's seat. Most pavers are equipped with dual controls so that the operator can sit near either side while he operates the paver.

It is impractical to describe in detail all tractor units in current use. There are several features, however, which are generally common to all and should be checked at the beginning of the paving operation and examined periodically thereafter. Most points that should be checked involve moving or working parts. For more detailed information, service manuals provided by the manufacturer should be carefully studied.

The governor on the engine should be checked for proper operation. It is important that the tractor unit provide a smooth steady pull on the screed arms. If the paver is equipped with pneumatic tires, air pressure should be as recommended. On crawler machines, crawlers should be snug but not tight. Any unnecessary movement caused by low tire pressure or loose crawlers when the machine starts or stops will be reflected in the surface of the mat as the screed is pulled forward.

Flow control gates at the back end of the hopper over each of the conveyor slats can be individualy adjusted to control the material flow to the spreading screws. Automatic controls actuate the spreading screws and the slat feeders to keep a constant depth of material in front of the screed.

The Screed Unit

The screed unit strikes off, partially compacts, and irons the surface of the mat as it is pulled forward. There is considerable variation as to detailed features and initial compaction principles. For these, service manuals and literature by the manufacturer should be studied.

Some screed units have a tamper bar which moves up and down and strikes off the paving mix so the screed plate can ride smoothly over it. The tamper bar also imparts most of the compaction to the mix in the spreading operation. More common are screeds that vibrate (Figure 6.6) but do not have tamping bars. Vibration may be accomplished with electric vibrators or eccentrically loaded turning shafts. The frequency of vibration may be controlled, thus helping to obtain a maximum compacting effort.

Both the leading and trailing edges of the screed have a crown adjustment. The leading edge should always have slightly more crown than the trailing edge to provide a smooth flow of material under the screed. Too much lead crown, however, creates an open texture along the edges of the mat. Crown adjustments may be made independently or simultaneously during the paving operation.

The screed is usually equipped with heaters to prevent the mix from sticking to the screed plate. They are used to heat the screed at the start of paving operations and sometimes when paving during cool, windy days. They should never be used, however, to attempt to heat the mix being delivered to the paver.

Fundamentals of Screed Operation

The screed unit, exclusive of any type of control, is attached to the tractor unit by two long arms that pivot about a point well forward on the paver. The basic principle of the screed is that when pulled into the material deposited in front of it by the spreading screws, it automatically rides up or down seeking the level where the path of its flat bottom surface is parallel to the direction of pull.

Forces acting on the screed unit during paving operations are shown in Figure 6.7. While the paver is moving, the pull, P, at the pivot point always exceeds the horizontal resistance, H, on the screed plate. When the thickness of the mat is to be increased, the screed is tilted upward to allow more material to crowd under it. The result is that the vertical uplift, V, exceeds the weight, W, and causes the screed plate to rise. As it rises, V becomes less until it again equals W, at which time the vertical motion stops and the screed plate once again moves only in the horizontal direction in a path

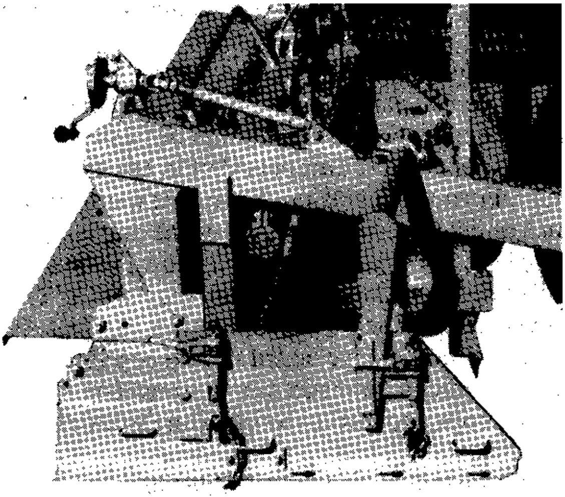


Figure 6.6 Vibratory Screed Unit.

parallel to the direction of pull. The thickness of the mat can be increased or decreased by tilting the screed plate up or down or by moving the pivot point of the pull arm vertically.

The screed is continually attempting to bring or keep all of the forces in balance. This is why it is important to set the flow gates properly, keep the slat feeders operating uniformly, keep a uniform height of material in front of the screed, and not over-control the screed. The temperature of the mix must be kept uniform so that the viscosity of the mix does not change and influence the balance of forces acting on the screed.

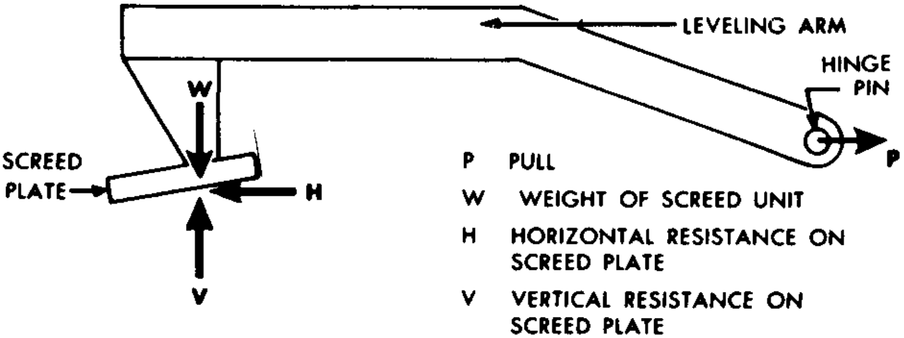

LEVELING ARM
HINGE PIN
W
SCREED PLATE
H
P
V
P PULL
W WEIGHT OF SCREED UNIT
H HORIZONTAL RESISTANCE ON SCREED PLATE
V VERTICAL RESISTANCE ON SCREED PLATE

Figure 6.7 Forces Acting on the Screed During Paving Operation.

Checking the Screed Unit

Many working conditions and adjustments can be checked only by the end result, or more specifically, by the quality of the mat that is placed.

Before work begins, however, checks should be made on items that would obviously affect the work.

The screed should be raised and the bottom surface checked for smoothness and excessive wear. Screed plates first wear out about 100 to 150 mm (4 to 6 in.) in from the trailing edge. Extensions should be flush with and in the same true plane as the bottom surface. Tampers should be checked for excessive wear, adjustment, and proper operation. Excessive wear causes a pitted surface in the mat, and improper adjustment gives the mat a scuffed appearance. If the screed is of the vibrating type, the vibrators should be started with the screed in a raised position to see that they work. Their performance should be checked during the paving operation.

A level mounted on the screed or on a wedge board should be available so that a check on the roadway crown can be made at any time. The heating unit should also be checked for proper operation by lighting the burner and allowing it to burn a few minutes.

An often overlooked but important item is the proper cleanup of the paving machine at the end of the working day. While the machine is still warm, the hopper, feeders, spreading screws, tamper bars, and screed plates should be given a light spray of petroleum distillate to ensure a smooth start the next day. Care should be taken to ensure that no accumulation of petroleum distillate is allowed, otherwise it could contaminate the mix when operations are started.

Automatic Screed Controls

Screed controls that can be adjusted for manual, semiautomatic, or automatic operation are available for almost every major make of asphalt paver. Automatic screed controls have five main components:

1. Sensor,
2. Pendulum,
3. Control box,
4. Command panel, and
5. Motors or cylinders to change the screed tilt.

A diagram showing the components of one make of automatic screed control is shown in Figure 6.8.

The screed control operates on the principle that if the screed is made to follow a smooth line regardless of irregularities in the surface being paved, a smooth pavement surface will result. Slope, or transverse profile, is controlled by a pendulum adjusted for a particular slope.

Once the screed is set for the desired depth of spread, the automatic system takes over to produce a smooth mat. The command panel, the grade sensor, and the pendulum feed electric impulses to a control box which activate the motors or cylinders to change the screed tilt, automatically compensating for road surface irregularities.

The sensor gets its information from a sensing device, or grid, riding on a stringline that has been set as the grade reference, or from a short or long ski riding on an adjacent lane, curb, or gutter. A stringline or long ski can be used for overlaying old pavements, but a carefully installed stringline is

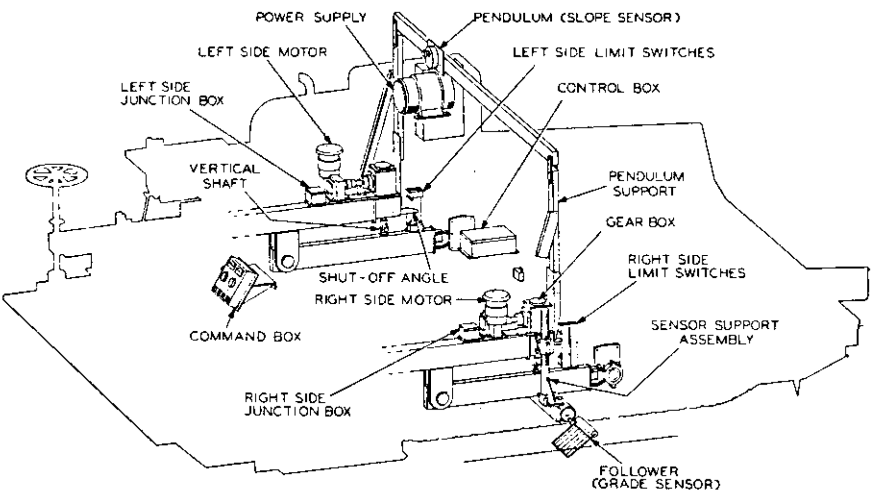


Figure 6.8 Components of an Automatic Screed Device.

recommended for new construction. A short ski or shoe can be substituted for the stringline or long ski after the first lane is placed. The stringline can be placed on either side of the paver, but the grade-follower skis must be mounted on the center-line side. Different types of grade followers are shown in Figures 6.9, 6.10, and 6.11.

Sensors on pavers equipped with electronic controls can be checked by varying the positions of the sensors and observing if the power on the screed controls responds and makes the correct compensating adjustment.

Small Self-Propelled Pavers

Most asphalt paver manufacturers make self-propelled machines for use on small jobs. The majority of these pavers have most, or all, of the features of their larger counterparts, including floating screeds (Figure 6.12).

In the past much of the hot-mix asphalt used for sidewalks, golf cart paths, jogging trails, etc. was placed by hand. This was partly due to the lack of appropriate equipment for mechanical placement and the inaccessible location of these sites. While many small pavers still range in length from 4 m (13 ft) to about 4.9 m (16 ft) there are at least two even smaller units. These are ideal for the small projects just mentioned. One that is less than 2.4 m (8 ft) long, is shown in Figure 6.13. It has no slat conveyor - the mix is dumped into a hopper located over the spreading screws - and the screed has neither tamping nor vibrating action. But, like the others, the screed's angle of attack can be changed to increase or decrease the pavement thickness. Also a gauge can be set to ensure minimum thickness.

An even smaller self-propelled paver is shown in Figure 6.14. On this unit the thickness of the mat is adjusted using cranks attached to the strike-off bar. The hopper of the unit holds less than a tonne (ton) of mix and is

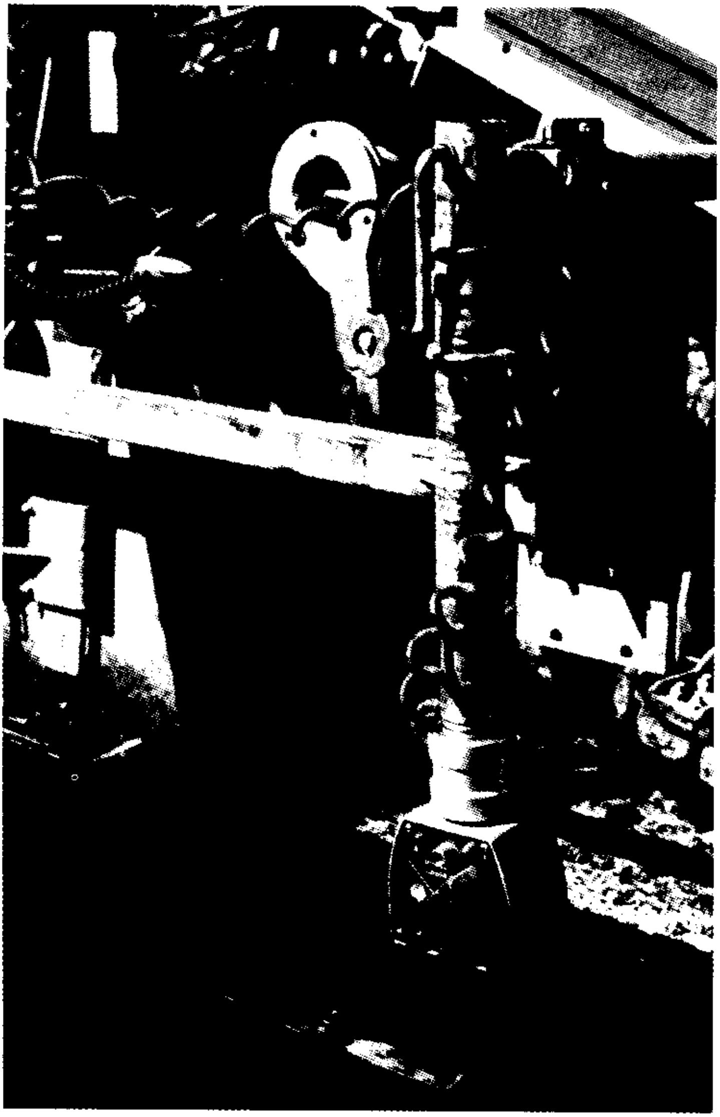


Figure 6.9 Short Ski or Shoe Grade Follower.

charged with a moveable conveyor or with a small front end loader if the dump truck cannot reach the hopper.

Towed-Type Pavers and Spreaders

On small paving jobs it is sometimes more convenient and economical to use a towed paver than to use the larger, self-powered pavers. Towed pavers are attached to the rear of the dump truck that hauls the asphalt mix from the plant. The material is deposited in the hopper of the paver and falls directly to the base. As the truck moves forward, the material is struck off by

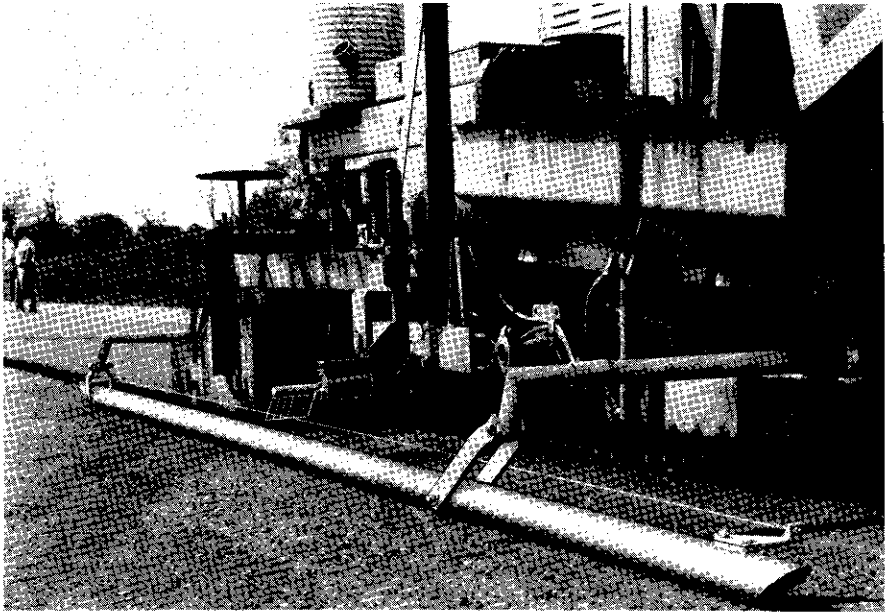


Figure 6.10 This Long Ski Grade Follower Was One of the First Automatic Screed Control Systems.

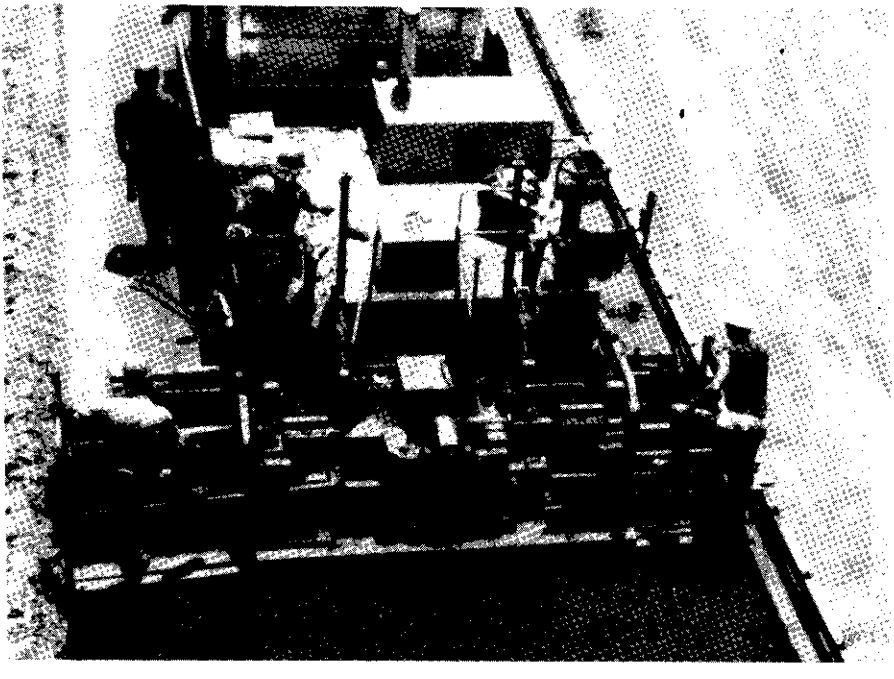


Figure 6.11 Grade Follower. Stringline Is Also Used as a Guideline for Paver Operator.

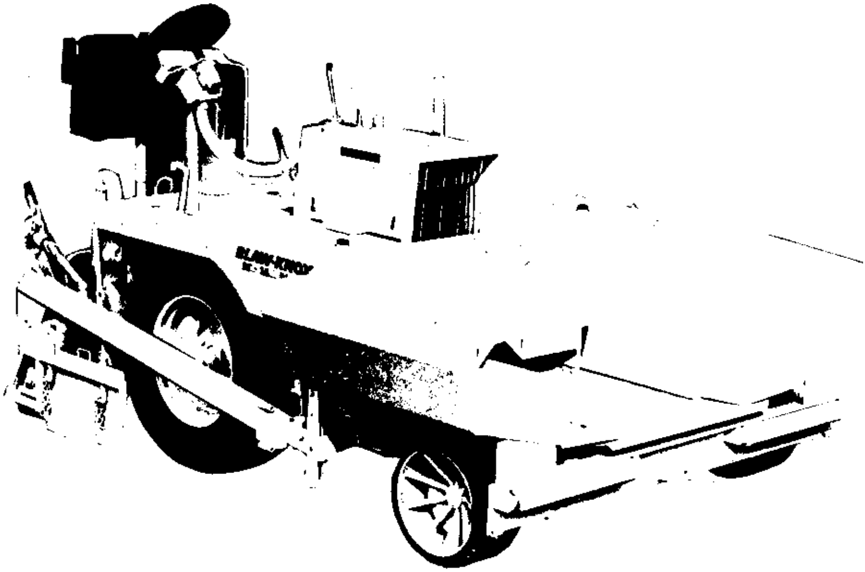


Figure 6.12 One Type of Small Paver. *(Courtesy Blaw-Knox)*

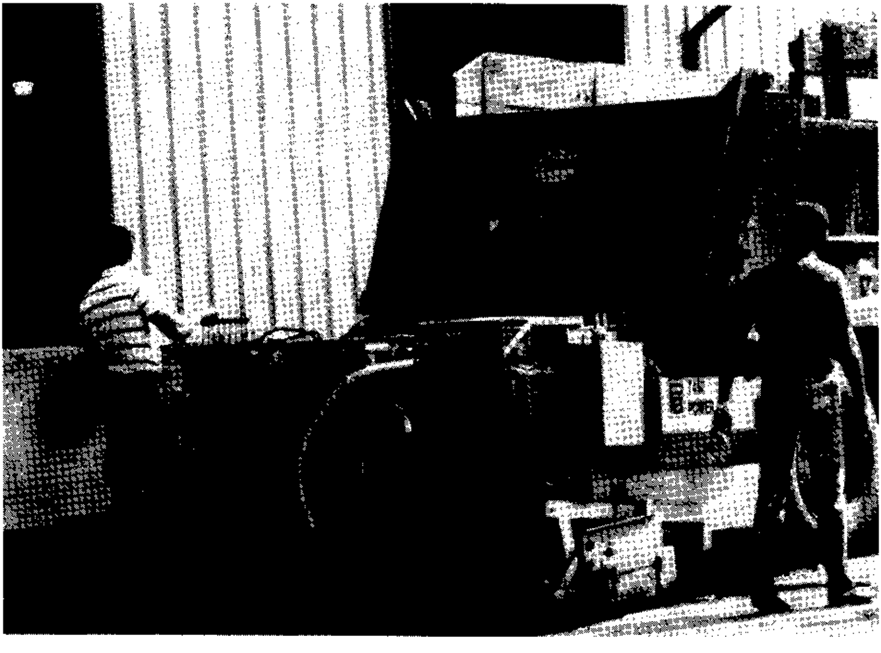


Figure 6.13 Self-Propelled Small Paver Less Than 2.4 m (8 ft) Long. *(Courtesy of Puckett Bros. Mfg. Co., Inc.)*

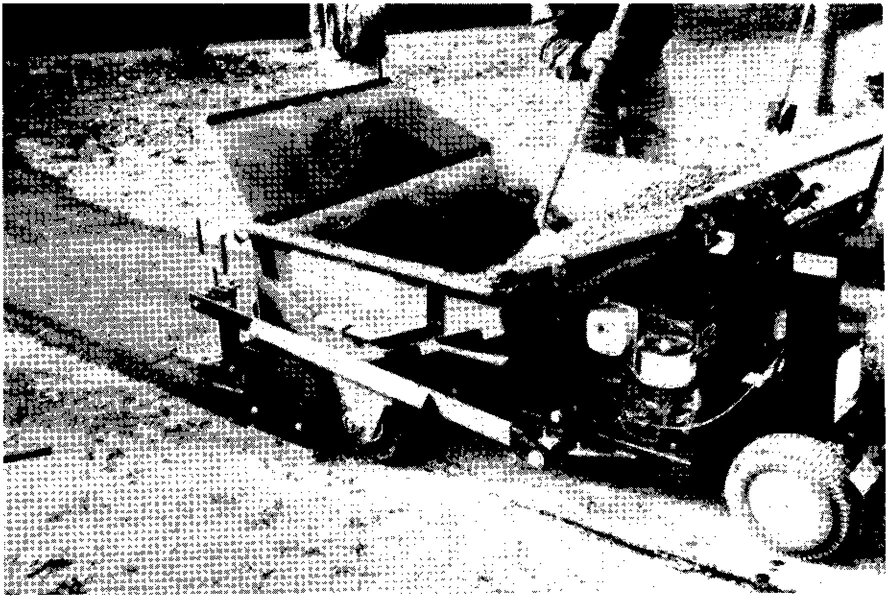


Figure 6.14 Small Self-Propelled Paver. *(Courtesy of Salsco, Inc.)*

a blade, a cutter bar, or by the screed and is ironed out by the screed or by rollers. Figure 6.15 shows the operating principles of one type of small paver. Screed heaters and screed vibrators are available as attachments for small pavers.

Many towed pavers have floating screeds. In order for the paver to start out spreading to full depth, blocking should be placed under the screed before any mix is dumped into the hopper. The hopper should be kept full of material during paving operations to assure a full, even spread. Once the screed is set for the proper depth it should not be necessary to adjust it again. If hot mix is being placed, the screed plate should be given time to warm by sitting on the mix before moving ahead.

The spreader should be towed at a uniform speed for any given setting of the screed or strike-off gate. If the towing speed varies, the spread will vary in thickness.

Auxiliary Equipment

Asphalt Distributor

The asphalt distributor is used to apply either a prime coat or tack coat to the surface to be paved. Prime coats are applications of cutback asphalt to an absorbent surface such as a granular base. Tack coats are very light applications of emulsified asphalt to an existing paved surface. Prime coats and tack coats are discussed in more detail in the chapter on surface treatments.

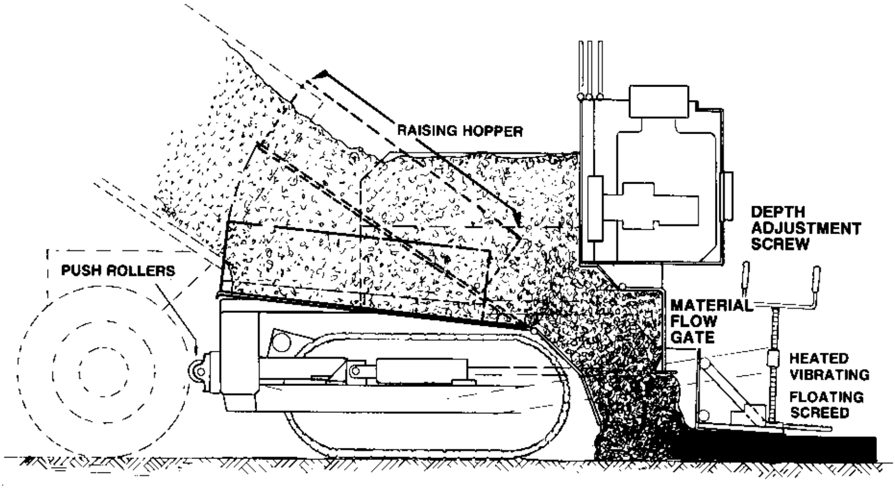


Figure 6.15 Flow of Material Through Paver. *(Courtesy of Puckett Bros. Mfg. Co., Inc.)*

The distributor consists of a truck or trailer on which is mounted an insulated tank. Some distributors are equipped with a heating system, usually oil burning (Figure 6.16). The distributor has a power-driven pump and a system of spray bars and nozzles through which the asphalt is forced, under pressure, onto the construction surface.

It is important that asphalt sprayed from a distributor be uniformly spread over the surface at the desired rate of application. This requires a pump in good working order, full circulating spray bars, and free flowing nozzles.

To obtain a desired rate of application, the speed of the distributor must be determined for a given pumping rate and coverage width. Distributors usually have a bitumeter which indicates the forward speed. This speed must be held constant if the asphalt coverage is to be uniform.

Motor Grader

In some situations the motor grader may be used to spread asphalt plant mixes, for example, in the placement of leveling courses. A leveling course is a thin layer of asphalt plant-mix placed under a pavement wearing surface. It helps provide a smooth, uniform wearing course by the paver.

Incidental Tools

Adequate hand tools and proper equipment for cleaning and heating them should be available for the paving operation. Incidental tools include:

1. Rakes;
2. Shovels;
3. Lutes;
4. Tool heating torch;

5. Cleaning equipment;
6. Hand tampers;
7. Small mechanical vibrating compactors;
8. Blocks and shims for supporting the screed of the paver when beginning operations;
9. Rope, heavy paper, or timbers for construction of joints at ends of runs;
10. Joint cutting and painting tools; and
11. Straightedge.

Not all of these tools are necessarily used on every paving project or every day of a particular job. Rakes, shovels, and lutes are frequently used by personnel around the paver. While the paver is operating, the laborers work or rework a portion of the mix to fit the paving around objects, or fill in areas that the paver does not pave adequately or cannot reach during the paving operation.

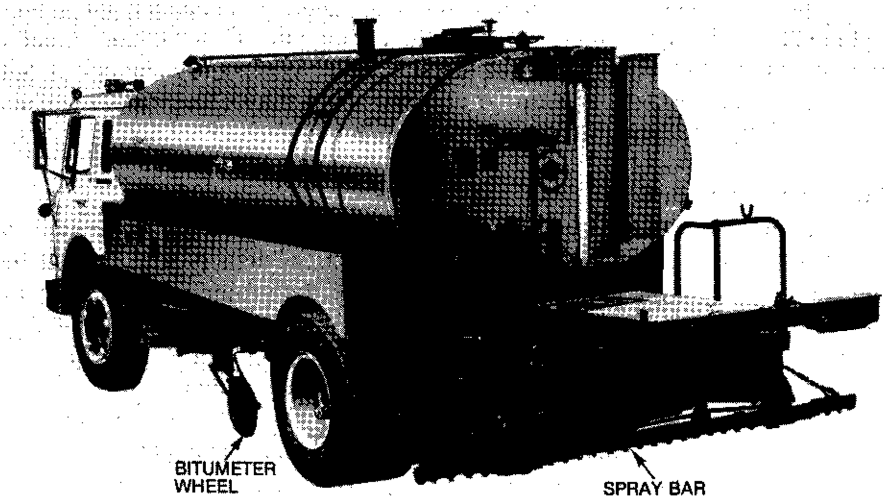


Figure 6.16 Asphalt Distributor.

6.2 Roadway Surface Preparation

Introduction

The Need for Roadway Preparation

A properly prepared foundation, which includes adequate surface and subsurface drainage and proper compaction during construction, will help to assure a long life for a pavement structure. Proper drainage is extremely important. The necessity for lowering the water table below the depth of frost penetration or the bottom of the improved subgrade by lateral drainage or subsurface drainage is generally accepted as an essential element of good construction by paving technologists.

Proper compaction of each layer of embankment, subgrade, improved subgrade, and base courses is equally important. Compaction greatly increases the supporting power of the subgrade. When the subgrade is not sufficiently compacted, additional consolidation may occur under traffic, with resulting settlement and possible failure.

Subgrade Compaction

Compaction tests are made in the laboratory on materials to be used in construction to determine the maximum practical density that should be obtained. These laboratory densities should be determined on the basis of the test: *Moisture-Density Relations of Soils Using a 10-lb (4.54 kg) Rammer and an 18-in. (457 mm) Drop;* AASHTO T 180, Method D. Recommended subgrade compaction criteria are as follows:

- For cohesive soils: minimum 95 percent density in top 300mm (12 in.) and minimum 90 percent for all fill areas below the top 300 mm (12 in.)
- For cohesionless soils: minimum 100 percent density in top 300 mm (12 in.) and minimum 95 percent for all fill areas below top 300 mm (12 in.).

The moisture content for compacting cohesive soils should be selected to provide the highest remolded strength consistent with expansion considerations. Generally, nonexpansive soils should be compacted to 1 or 2 percent on the dry side of laboratory optimum moisture content for best results.

The compaction criteria should also apply to cut sections. In some instances this requires scarifying, adding moisture, replacement, and recompaction to produce uniformity in the subgrade and avoid differential settlement.

Although compaction increases the stability of most soils, some soils decrease in stability when scarified, worked, and rolled. There are also some soils that shrink excessively during dry periods and expand excessively when allowed to absorb moisture. When these conditions exist, special treatment is required under the supervision of a soils engineer. Such conditions include clays that lose strength when remolded, silts that become "quick" when remolded, and soils with expansive characteristics.

Proof Rolling

Surface irregularities in asphalt pavement structures most often result from traffic densification of the underlying courses, usually in the wheel paths. Such irregularities indicate that one or more courses were not adequately compacted during construction, or that soft, unstable areas exist in the structure. To detect these deficiencies proof rolling is sometimes performed during construction.

Proof rolling is the application of heavy, pneumatic-tired rollers over the course, usually following initial compaction by conventional means. Proof rolling accomplishes two purposes: (a) it locates unstable areas, and (b) it achieves additional compaction.

Preparation of Unpaved Surfaces

General

The following roadway surfaces are generally considered unpaved surfaces:

1. Compacted subgrade,
2. Improved subgrade,
3. Untreated base, and
4. Non-surfaced granular roadway.

Certain treated or stabilized granular bases are considered unpaved surfaces when being prepared for asphalt paving. Bases that have been treated or stabilized with either asphalt or portland cement are considered paved surfaces and are excluded from this classification.

Prepared Subgrade

A prepared subgrade is one that has been worked and compacted. This may be the foundation soil or a layer of stabilized soil, select soil, or otherwise improved subgrade material.

The riding quality of the pavement surface depends largely on proper construction and preparation of the foundation material. The roadway should be shaped and rolled so that the paving equipment has no difficulty in placing the material at a uniform thickness to a smooth grade.

Weather conditions should be suitable and the roadway surface should be firm, dust free, and dry, or but slightly damp when paving operations are started.

Untreated Base

Untreated granular bases and some chemically stabilized bases other than portland cement and asphalt-treated bases should also be properly shaped and compacted.

When asphalt pavement courses are to be placed on an untreated aggregate base, loose granular particles should be swept from the surface of the roadway using power brooms. Care should be taken, however, not to dislodge or otherwise disturb the bond of the aggregate in the surface of the base. When the loose material has been properly removed, only the tops of the aggregate in the surface will be exposed. When the surface has been cleaned it is ready to be primed with asphalt.

In priming, a pressure distributor is used to spray, under average conditions, from 0.9 to 2.3 litres/m^2 (0.20 to 0.50 gal/yd^2) of low viscosity asphalt, such as MC-30 or MC-70 on the prepared surface of an absorbent base. (On highly absorbent bases, MC-250 is sometimes used.) The asphalt then penetrates into the surface. The asphalt should be entirely absorbed by the base course. If it is not absorbed within 24 hours after application, sand should be spread over the surface to blot the excess asphalt. Care should be exercised, however, to prevent overpriming. The prime should be fully set and cured before placing the hot-mix asphalt.

If MC cutback asphalt is not available, SS-1, CSS-1, SS-1h or CSS-1h emulsified asphalt may be used as a mixed-in prime, either on the road or at the source of base aggregate. If a layer of base aggregate is to be supplied, the emulsion and compaction water can be mixed at the source. The application rate is normally 0.5 to 1.4 litre/m^2/25 mm (0.1 to 0.3 gal/sq yd/in.). Under most circumstances it is more difficult for the emulsified asphalt to penetrate a granular base than for the cutback asphalt. Therefore, the cutback asphalt should be used when possible.

Non-Surfaced Granular Roadways

Non-surfaced granular roadways are similar to untreated granular bases. They differ primarily in that they have been used as a roadway by traffic and, typically, have been in use for a considerable time.

Again, it cannot be stressed too strongly that the riding quality of the surface depends to a great degree on conditioning and preparing the underlying pavement structure. It is advisable, where the granular surfaced roadway is rough and uneven, to bring the surface to grade by scarifying the top few inches of material, blending in more material as may be necessary, compacting, and priming with asphalt. If the granular roadway has had a previous asphalt treatment and is rough and uneven, it is advisable to place a leveling course of hot-asphalt plant-mix ahead of the first layer of the surface course. Asphalt leveling courses will be discussed later in this chapter. The engineer should make an inspection of the surface well in advance of placing the surface courses and determine what corrections need to be made.

Free and excessive screenings and loose material should be removed from the roadway and the surface primed with asphalt before placing any leveling or asphalt surface courses.

Preparation of Old Pavement Surfaces

Old Asphalt Mix Pavements

Structural distress occurring in old asphalt pavements is usually the result of inadequate design, inadequate execution of the design including compaction, or both. Poor mix design can also cause several types of distress, and excess asphalt may cause corrugation or rutting. Cracking may be caused by excessive pavement deflection under traffic due to an inadequate pavement structure or a spongy foundation. Insufficient or oxidized asphalt may also contribute to cracking because the mix may be brittle.

Before the old pavement receives an overlaying course of hot-mix asphalt, it should first be inspected. The remedy for whatever failures exist depends on the type and extent of the distress. If failure is extensive, reconstruction will probably be necessary. In any case, all needed repairs to the old surface should be made before paving.

A slick surface on an old pavement may be caused by aggregate polishing under traffic or by too much asphalt in the mix. If the cause is excess asphalt, it should be removed with a heater-planer or cold-planer before placing the overlay.

Old pavements having small cracks should be given a fog seal before overlaying. A fog seal is a light application of emulsified asphalt diluted with water and sprayed over the surface. Larger cracks warrant having the surface treated with an emulsion slurry seal. The slurry seal is a mixture of emulsified asphalt, fine aggregate, and mineral filler, with water added to produce a slurry consistency. These treatments are described and discussed later in greater detail in Chapter 8.

Pot holes in an old pavement that is otherwise strong should be patched with asphalt concrete before the roadway surface is paved. If the base beneath the old pavement has failed, damage will be reflected in the new pavement surface. Patching, of course, must be done before paving, and the patch should be deep enough to strengthen the base.

All bleeding and unsuitable patches, excess asphalt crack or joint sealer, loose scale, and any surplus asphalt material should be removed from the surface of the existing pavement.

All depressions of 25 mm (1 in.) or more should be overlaid with a leveling course and compacted ahead of the surfacing operation. All surfaces, both horizontal and vertical, that will be in contact with the new asphalt surface must be thoroughly cleaned. Cleaning flat surfaces is usually done with rotary brooms, but washing or flushing may be necessary to remove clay or dirt. A tack coat, discussed later, should be applied to the existing pavement, and to all vertical faces. A uniform coating of asphalt emulsion helps provide closely-bonded waterproof joints.

High spots can be removed by a heater-planer, a machine that heats the old pavement surface and then planes it off to a predetermined depth. Cold milling machines do the same job without first heating the pavement surface (Figure 6.17).

There are both large and small milling machines available today that can remove asphalt concrete pavement from 3 metres (9 feet) widths down to 0.3 metre (1 foot) and in lift thicknesses between 25 to 100 mm (1 to 4 in.) in one pass. The widths and thicknesses are approximate and depend on equipment manufacturers and conditions of roadway. The benefits of using a milling machine can be numerous depending on the design features of the project. However, the minimum benefits are:

1. It generates a fairly uniform material that can be utilized for recycling in new hot mixes or other recycling or reclamation uses.
2. It can sustain grade on a structurally sound or strong pavement where just the surface can be removed and replaced.
3. It often can do a better job of leveling and truing because it can remove a variable thickness of material more confidently than the paver can sometimes level and true a pavement.

Therefore, milling should always be considered as a possible option in the rehabilitation process. When a surface has been milled, the surface should be swept as clean as possible, with the sweepings discarded and not used as a portion of the recycled aggregate. Even though the surface leaves a very coarse herringbone type texture, the milled surface should always be tack coated prior to overlay.

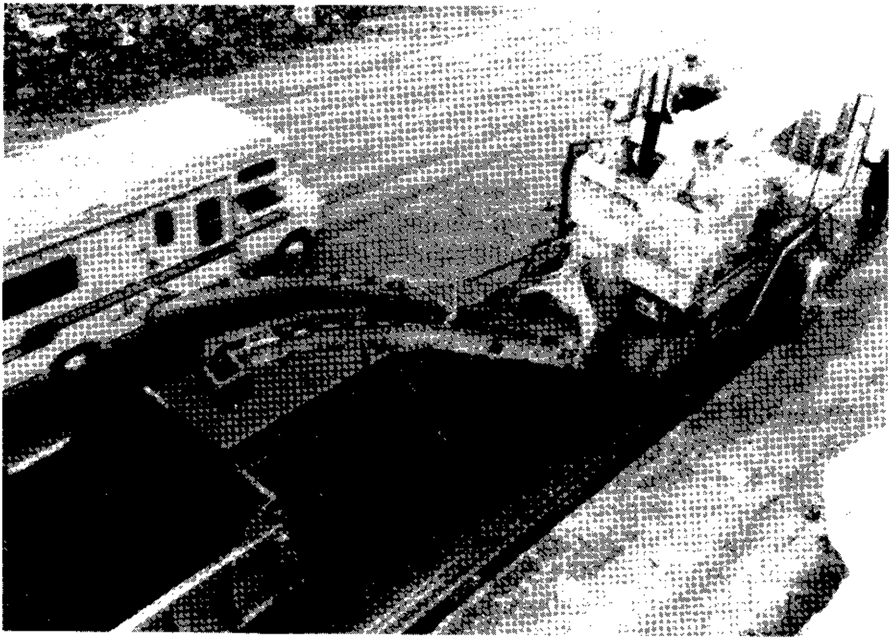


Figure 6.17 Cold Milling. *(Courtesy of CMI Corporation)*

Rigid-Type Pavements

Distress in rigid-type pavements also must be corrected before resurfacing. Additional layers of asphalt mixtures must be thick enough not only to provide required additional strength but also to minimize the reflection of cracks from the old pavement to the new surface.

Preparation of damaged or distressed rigid pavements differs from that for asphalt pavements. Preparation may include one or more of the following:

1. Patching disintegrated and spalled areas.
2. Sealing cracks to prevent the intrusion of water.
3. Undersealing to provide uniform support.
4. Cracking slabs that rock under traffic and seating them with heavy rollers.

Pumping at the joints is caused by the rocking of slabs under traffic. The wet subgrade is forced from beneath the slab and this causes cracking and breaking in the joint area. Usually a pavement can be stabilized by undersealing with asphalt specially prepared for this purpose. Otherwise, the rocking slabs should be cracked into smaller pieces and seated.

In some cases portions of the pavement may be so disintegrated or badly broken up that the pavement fragments should be removed. These areas should be prepared and patched with asphalt concrete prior to the resurfacing operation.

Filler material should be removed from joints and cracks in the slabs to a depth of at least 6mm ($^1/_4$ in.). The joints should be refilled with an asphalt joint sealing material. Any asphalt patches having excess asphalt, as well as any excess crack sealer that may have accumulated next to cracks and joints, should be removed.

As a final step, before any asphalt paving is placed, the pavement surface should be swept clean and given a tack coat. Where preparation measures have caused settlement or pavement roughness, a leveling course of asphalt concrete should be placed prior to the resurfacing operation.

Consideration should be given to the use of some type of crack relief system over all joints when a relatively thin lift [less than 25mm (4 in.)] is to be placed.

Tack Coats

A tack coat is the application of asphalt (usually emulsified asphalt) to an existing paved surface to ensure bond between the old surface and the asphalt overlay. The three essential requirements of a tack are it must be very thin, it must uniformly cover the entire surface of the area to be paved, and it must be allowed to break (cure) before the hot-mix asphalt is applied.

To accomplish this, type SS-1, SS-1h, CSS-1 or CSS-1h asphalt emulsion is diluted with an equal amount of water. The rate of application should be such that about 0.25-0.70 litre/m^2 (0.05-0.15 gal/yd^2) of the diluted material covers the surface.

Tack coats having a surplus of asphalt over the surface sometimes form a slip plane. Excessive asphalt may also work its way up into the overlying course, which may eventually flush asphalt to the surface. A light tack coat does no harm and it bonds the surfaces, a necessity for good construction.

Work should be planned so that no more than the necessary tack coat for the day's operation is placed on the surface. All traffic not essential to the work should be kept off the tack coat.

Vertical faces of existing pavements such as curbs and gutters should also be sprayed or painted with a uniform coating of asphalt emulsion. This work should be done so that exposed curb or gutter surfaces are not stained.

Leveling Courses

Purpose of the Leveling Course

Placing a leveling course is an operation employed when the road surface is so irregular that it exceeds the leveling capabilities of the paver. The paver is usually effective on irregularities of lengths not longer than 1½ to 2 times the wheel base of the machine. Beyond these lengths, a grade control device is necessary. Generally, irregularities no longer than 12 or 15 m (40 to 50 ft) can be handled with a traveling stringline device and automatic screed control on the paver.

The existing center-line profile, as determined by preliminary measurements and observations, should be viewed and evaluated. Based on this information, the decision should be made whether the entire project, portions of the project, or none of it is to undergo the leveling course operation. This decision should be made on the basis of condition of the existing surface for overlays and the surface of the first course of asphalt concrete on a new grade.

Spreading with a Motor Grader

Because a high standard of smoothness in all layers of the asphalt pavement structure is required by modern high-speed traffic, long wheelbase motor graders having smooth-tread tires often are used for spreading hot-mix asphalt in leveling courses. In laying the leveling course with a blade, the principal advantage offered by the longer wheel base of the grader is the elimination of excessive humps, sags, and irregularities in the subgrade or old pavement on which the mix is being placed. Another benefit realized is a rough textured surface on which to place the overlying course.

In placing a course of hot-mix asphalt with a motor grader, a roller is needed to compact the mix closely behind the grader. It is preferable to use a pneumatic-tired roller in conjunction with the blade in this type of operation. With each pass of the motor grader, the roller should follow immediately, compacting the spread that is laid. This operation should progress until the final desired surface evenness is reached. If the motor grader operator manipulates his controls properly he will leave more material in the depressions and less material at the humps, and the roller immediately behind will compact this material, gradually building up a smooth, firm plane on which to place the succeeding course or courses.

In placing the mix in a leveling course by motor grader, it is essential that, in each station of the project, there not be a large excess of surplus material. The motor grader is not designed to carry a considerable quantity of mix much distance. Its primary function is to spread the required amount laterally, moving longitudinally only enough to smooth out irregularities.

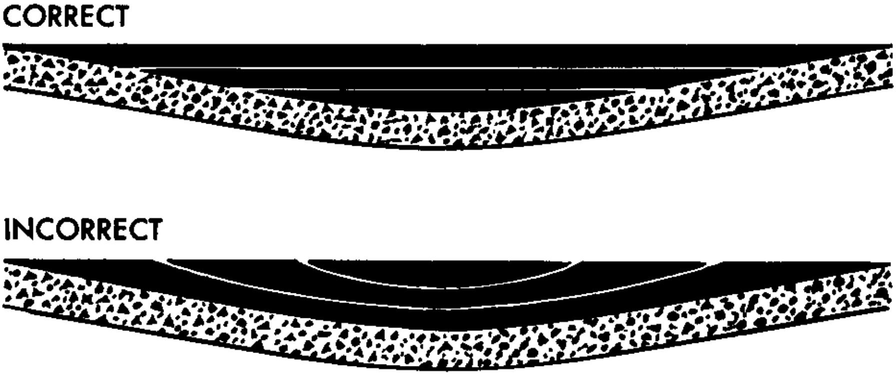
A long stringline may be stretched over the surface to locate the high points on the existing pavement surface. They are then marked and are projected to stakes at the side of the road with a carpenter's level and straightedge, or with a stringline level. After enough stakes are set at the side, a line is tightly drawn through the points marked on the stakes. Intermediate stakes should be set to take the sag out of the line. Then by sighting over this line supported by the stakes, abrupt profile changes can be "eyed out." When the line is satisfactorily smoothed out for profile, a constant can be determined that will give the desired minimum cover over the high spots, thus fixing the grade for the leveling course.

If time permits, a more accurate method of determining the amount of mix required per station is by calculations from level notes.

Leveling Wedges

Leveling wedges are patches of asphalt plant-mix used to level sags and depressions in an old pavement prior to the surfacing operation. Placing leveling wedges is part of the leveling course operation.

Leveling wedges should be installed in two layers if they are from 75 to 150 mm (3 to 6 in.) in thickness. Wedges thicker than 150 mm (6 in.) should be placed in compacted layers of not more than 75 mm (3 in.). In placing multiple layers, the layer of the shortest length should be placed first (the lowest portion of the depression), with successive layer(s) extending over or covering the short ones (Figure 6.18). If placed incorrectly, there would be a tendency for a series of steps to develop at each joint because of the difficulty of feathering out asphalt mixtures at the beginning and end of a layer. A bump would likely develop and be reflected in the final surface.


CORRECT

INCORRECT

Figure 6.18 Leveling Wedge Layers for Leveling Dips and Depressions.

Where wedging of dips and depressions requires multiple layers, the engineer should have sufficient levels taken to plot profiles and cross sections accurately. From these plots, the grades of the proposed correction and the linear limits of the successive layers should be determined so that definite stationing for starting and stopping the spreader or motor grader passes can be decided (Figure 6.19).

In some instances, where time does not permit making a preliminary survey, a flat line can be erected if care and judgment are exercised. The line is fastened tightly to the surface and visually straightened by shimming to remove the sag. The line either erected or shimmed can be made a reference line for a paver having automatic screed controls for placing the leveling wedges.

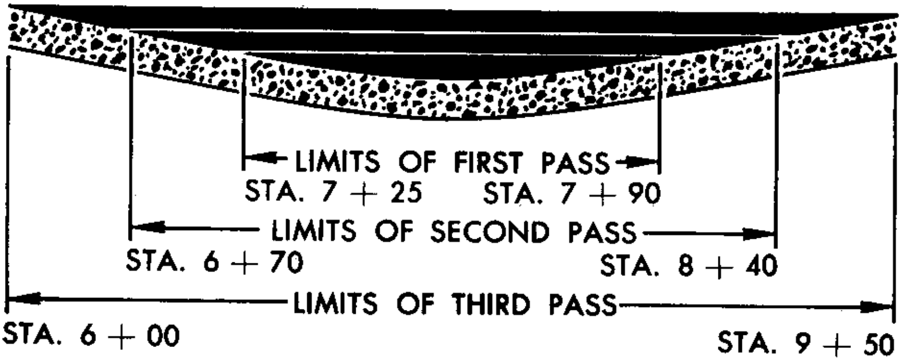


Figure 6.19 Limits for Multilayer Leveling Wedges Stationed for Spreading.

6.3 Placing Hot-Mix Asphalt

Introduction

Engineer-Contractor Relationships

Modern practice has established that a dual system of construction control results in a good working relationship. The engineer designs the project and prepares the plans, specifications, and estimates. Contractors bid competitively and supervise the construction, with the work subject to inspection and acceptance by the engineer. This dual control system of supervision and inspection makes maximum cooperation between the engineer and contractor's forces essential. Each must recognize and respect the position of the other. Although the plans and specifications should clearly define the end product required by the owner, they should not be too restrictive as to methods and equipment that might be used to accomplish the desired results more efficiently.

Planning Spreading (Placing) Operations

Before spreading operations, the engineer and his inspector should confer with the contractor's superintendents and foremen and carefully plan the entire operation. Some important details that should be discussed are:

1. Continuity and sequence of operations,
2. Number of pavers needed for the project,
3. Number and types of rollers needed,
4. Number of trucks required,
5. The chain of command for giving and receiving instructions,
6. Reasons for possible rejection of the mix,
7. Weather and temperature requirements, and
8. Traffic control.

Before spreading with a paver is begun, agreement on a number of items must be reached. One is the placing temperature of the mix. In general, the temperature as the mix is placed on the road should be at the minimum for best workability and compaction. (See Chapter 7).

The spreading rate, kg/m^2 (lb/yd^2) or the thickness of the course should be agreed upon. The paving sequence and pattern should be established and understood.

If possible, it is always desirable to operate two pavers in echelon or use a full-width paver. When two pavers are used, they should be kept within 15 m (50 ft.) of each other. The object is to maintain a hot joint between the two spreads. Other factors affecting the joint are air temperature, mix temperature, and temperature of the surface on which the mix is being placed.

Before Paving Starts

Before the paving operation begins, the technician should make sure that all necessary inspections and preparations have been made. The following items are included:

1. Base or roadway surface properly prepared;
2. Plan for execution of the work;
3. Proper balance of mix production, spreading, and rolling;
4. Equipment in good condition and adjustment;
5. Provisions for weighing the mix; and
6. Plan for sampling and testing.

Receiving the Mix

Load Tickets

A routine task of the inspector is to collect load tickets (Figure 6.20) from truck drivers at the point of delivery. Tickets indicating the amount of hot-mix asphalt are presented to the truck driver at the hot-mix asphalt mixing facility. Collection of the tickets assures the inspector that the material represented by the load ticket is actually incorporated into the work. An accurate record of all load tickets minimizes the possibility of any loads being diverted from the job. They are also used as a check against plant records for the day's total tonnage. The inspector should personally collect all load slips from truck drivers and not allow anyone else to collect them, except when he is absent from the paving operation.

With the collection of load tickets and the desired spread in kilograms per linear meter (pounds per linear foot) or tonnes (tons) per station, the average spread per load can be checked. This check is of greater importance when asphalt concrete is placed in a windrow before spreading than when it is dumped directly into a paver.

Only by knowing what the desired spread is, and seeing that the mix is placed in that amount, can the inspector be sure that he will not excessively over or under run the contract quantities.

Inspection of Mix

Close cooperation between the paving crew and the asphalt plant is essential in securing a satisfactory and uniform job. A fast means of communication should be established between the paving operation and the asphalt plant so that any change in the mixture can be made promptly. When possible, the paving inspector and the plant inspector should frequently exchange visits. When the paving inspector is familiar with mixing facility operations, he can easily determine if mix changes are necessary. The plant inspector, on the other hand, by being familiar with the paving operation can better understand problems attendant to it.

Every truckload of material should be observed as it arrives. The mix

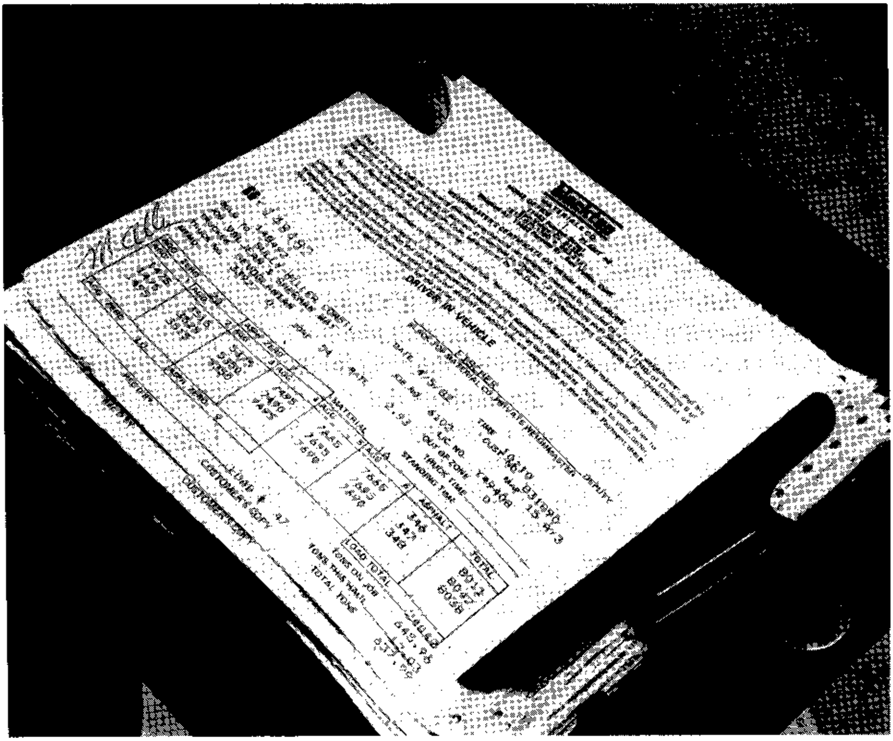


Figure 6.20 Illustration of Load Ticket.

temperature should be checked regularly. If it is not within specified tolerance, the mix should not be used.

Mistakes in batching, mixing, and temperature control can and do occur, and these errors may sometimes go unnoticed by the plant inspector. Consequently, loads arriving at the spreader may be unsatisfactory. When the paving inspector rejects a load he should record his action, with the reason for rejection, both on the ticket and in his diary so that the proper deduction can be made from the pay quantities. If appropriate, a sample should be obtained for laboratory analysis. A record should also be kept of the loads accepted and placed. These records should be checked daily, or more frequently, with those of the plant inspector so that discrepancies do not exist when work is completed.

Mix Deficiencies

Some mix deficiencies that may justify discarding the mix include:

1. *Too Hot*—Blue smoke rising from the mix usually indicates an overheated batch. The temperature should be checked immediately. If the batch exceeds maximum specification limits, it should be discarded. If it exceeds optimum placing temperature but does not exceed the specification

limit, the batch is usually not discarded, but immediate steps should be taken to correct the condition.

2. *Too Cold*—A generally stiff appearance, or improper coating of the larger aggregate particles, indicates a cold mixture. Again the temperature should be checked immediately. If it is below the specification limit, it should be discarded. If it is within the specification limit but below optimum placing temperature, steps should be taken immediately to correct the situation.
3. *Too Much Asphalt*—When loads have been arriving at the spreader with the material domed up or peaked and suddenly a load appears lying flat, it may contain too much asphalt. Excessive asphalt may be detected under the screed by the way the mix slicks off.
4. *Too Little Asphalt*—A mix containing too little asphalt generally can be detected immediately if the asphalt deficiency is severe. It has a lean, granular appearance, improper coating, and lacks the typical shiny black luster. The pavement surface has a dull, brown appearance and the roller does not compact it satisfactorily. A less severe deficiency is difficult to detect by appearance; suspicions should be checked by testing.
5. *Nonuniform Mixing*—Nonuniform mixing shows up as spots of lean, brown, or uncoated dull-appearing material within areas having a rich, shiny appearance.
6. *Excess Coarse Aggregate*—A mix with excess coarse aggregate can be detected by the poor workability of the mix and by its coarse appearance when it is on the road. Otherwise it resembles an over-rich mix.
7. *Excess Fine Aggregate*—A mix with an excess of fine aggregate has a different texture from a properly graded mix after it has been rolled. Otherwise it resembles a lean mix.
8. *Excess Moisture*—Steam rising from the mix as it is dumped into the hopper of the spreader indicates moisture in the mix. It may be bubbling or popping as if it were boiling. The mix may also foam so that it appears to have too much asphalt.
9. *Miscellaneous*—Segregation of the aggregates in the mix may occur because of improper handling and may be serious enough to warrant rejection. Loads that have become contaminated because of spilled gasoline, kerosene, oil, and the like, should not be used in the roadway.

Placing With The Paver

Guide Line

A guide line for the paver operator should always be used. It should be parallel to the center line of the road and located along the edge or near the longitudinal joint of the lane being paved. Unless the guide line is a supported stringline for a paver having an automatic grade control, it is not necessary to stretch it any tighter than is necessary to make it straight.

The guide line should be marked or staked out at least 150 m (500 ft) in advance of operations to avoid delays. On tangents where line stakes are used, they should be placed at intervals of not more than 30 m (100 ft). On curves, they should be placed at intervals of 7.5 m (25 ft) or less.

Supported Stringline

When a line is erected as a grade reference, it must be carefully located and stretched tight. It acts as the grade line that the sensor feels to control the screed. The height of the stringline can be raised or lowered to accommodate different ground-level conditions.

The stringline may be a good grade of baler twine or heavy cord. It must be drawn taut and anchored in equal distances 90 to 120 m (300 to 400 ft.), unless the specifications require shorter distances. When resurfacing existing roads or streets, the string can be stretched from high point to high point and anchored with nails.

Width of Spread

Width of spread is controlled in many instances by the location of the longitudinal joint. When more than one pavement course is placed in the roadway, the longitudinal joint should be offset at least 150 mm (6 in.) from the longitudinal joint of the lower pavement course (Figure 6.21). Overlapping of successive courses prevents cracking along the longitudinal joint.

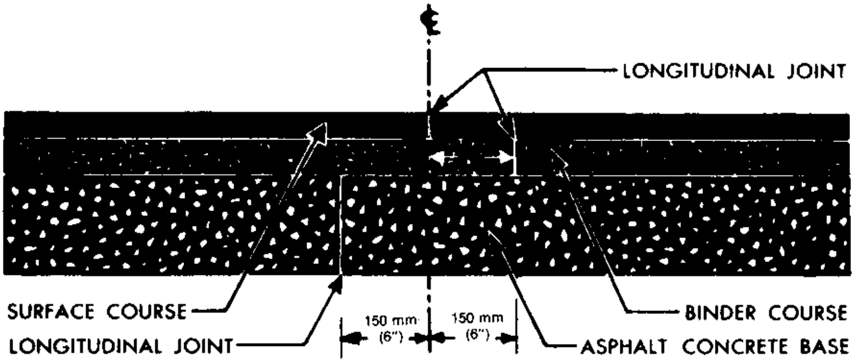


Figure 6.21 Overlapping of Successive Courses to Help Prevent a Crack Opening Along a Longitudinal Joint.

Rate of Spread

For a given width, thickness, and density, the length of pavement per unit of weight of paving mix can be determined. The inspector or technician may check the actual distance paved against that determined for a given load, and decide whether a screed adjustment is necessary. One means of determining the length of spread per unit of weight is by:

Metric Units	Customary Units	
$L = \frac{1000}{AW}$	$L = \frac{2000}{AW}$	(1)

where	where
L = linear m per tonne	L = linear ft per ton
A = cross-sectional area (width × thickness) sq m, and	A = cross-sectional area (width × thickness) sq ft, and
W = compacted density, kg per cu m.	W = compacted density, lb per cu ft.

Some paving is placed on the weight per unit area basis. In this case, for a given width of paving course the length per unit of weight may be calculated:

Metric Units	Customary Units	
$L = \frac{1000}{rW}$	$L = \frac{18{,}000}{rW}$	(2)

where	where
r = spreading rate, kg per sq m, and	r = spreading rate, lb per sq yd, and
W = width of spread, m.	W = width of spread, ft.

Similar formulas may be developed to best serve the inspector's or technician's purpose. Tables have been published for field use, but the inspector or technician may choose to develop his own tables.

The rate of spread may also refer to the forward speed of the paver. For best results, the rate of plant production and the rate of placing material should balance. The required forward speed to balance with plant production may be found by:

$$V = \frac{R_p L}{60} \qquad (3)$$

where

V = forward speed of paver m (ft) per min;
R_p = rate of plant production, tonnes per hr (tons per hr); and
L = linear m per tonne (linear ft per ton) of mix, from Eq. 1 or 2.

However the paving machine should never be operated at a speed in excess of that which will result in a properly placed mixture. If the paver cannot properly place the mixture at a rate equal to plant capacity, the plant production rate will need to be slowed, or other corrective measures taken.

The Placing Operation

When the paver is positioned on the road, the screed should be lowered on to blocks having the same thickness as the loose mat to be laid down by the paver. The thickness control screws on the screed are then adjusted for this depth. A rule of thumb for determining the thickness of the loose mat is to multiply the desired compacted thickness by 1.25.

When starting from the end of a previously laid mat, boards having a thickness equal to the difference between the loose and compacted layers should be used. The paver should then begin spreading at the proper full depth of the uncompacted material.

As the paver spreads its first load of asphalt mix, the mat should frequently be "stabbed" or checked for thickness (Figure 6.22). After 6 or 9 m (20 or 30 ft) of operation the paver should be stopped and the new surface checked. The texture of the unrolled surface should be uniform. If it is not, screed adjustments may be necessary. The adjustment of the screed, tamping bars or vibrators, spreading screws hopper feed, and other adjustments should be checked frequently to assure uniform spreading of the mix to proper line and grade.

Segregation of materials must not be permitted. If it should occur, the spreading operation should be stopped immediately and not resumed until the cause of mix segregation is determined and corrected.

When the truck is dumping its load into the hopper, its wheels should remain in constant contact with the truck rollers of the paver. If a truck is skewed in such a way that the rollers are pushing against one set of dual wheels only, the spreader tends to skew. In this case, the operator has to make corrections continually, resulting in a ragged line with consequent irregular and poorly compacted joints.

The amount of material carried ahead of the screed should be kept uniform in height. Variation may result in surface roughness. This will require maintaining sufficient material in the hopper to supply the spreading screws with enough mix to cover at least 2/3 of the depth of the screws out to their ends. Control gates in the hopper should be set so the screws and feeders operate 85 percent or more of the time.

Asphalt mix that clings to the sides of the hopper should be continually loosened and pushed into the active mix. If the mix is permitted to accumulate, it cools rapidly, and eventually a slug of semi-cold mix reaches the roadway. This may result in a nonuniform surface texture and an area prone to ravelling.

Automatic Screed Operation

The sensor riding on an erected stringline transmits signals to the screed control to produce a paved mat behind the screed at a predetermined grade. The cross slope of the screed is normally set on the control panel.

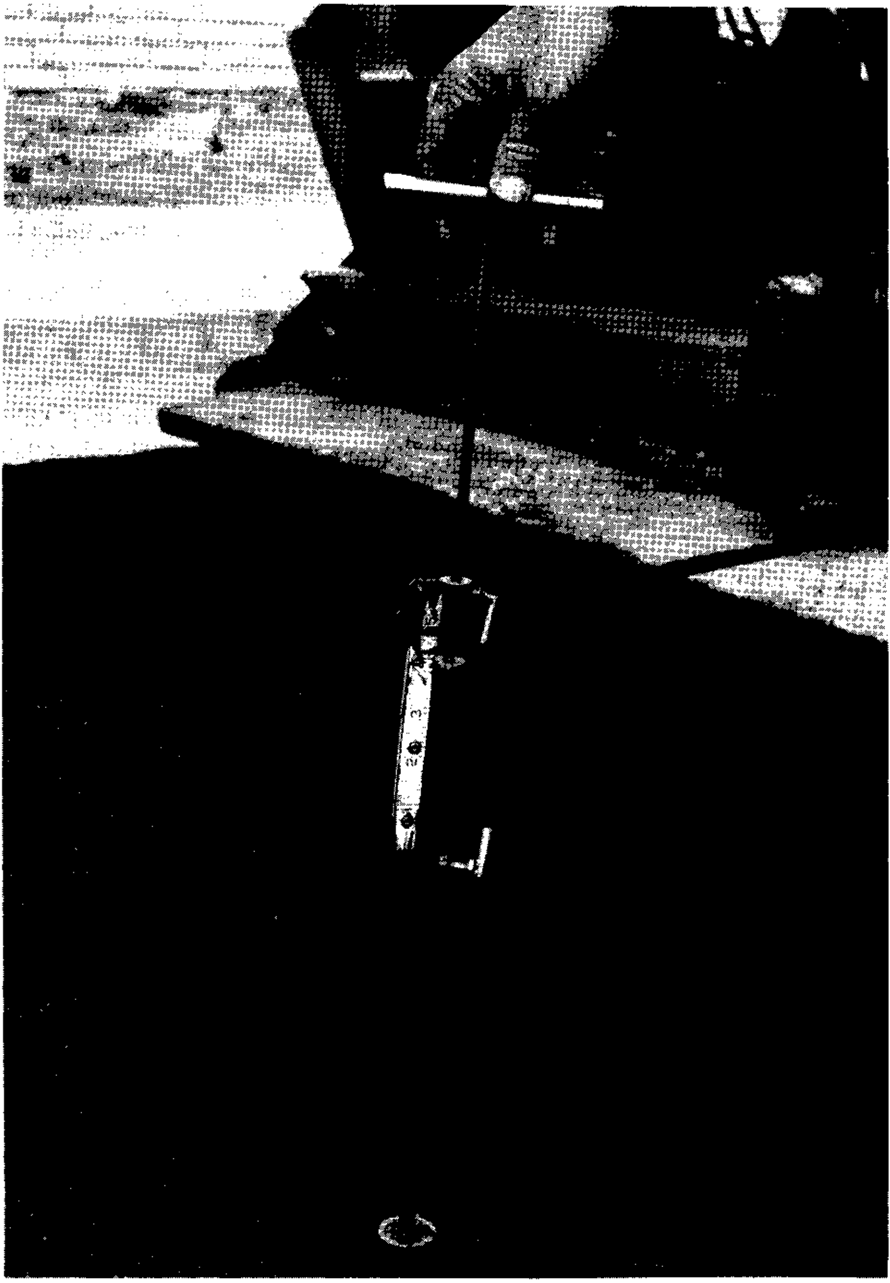


Figure 6.22 Checking Mat Thickness With a Depth Gauge.

When a sensor or grade follower is linked in with a short ski, long ski, or other traveling reference device, the average thickness may be adjusted by raising or lowering the sensor. The average thickness may be checked by determining the length of spread per unit of weight as explained earlier.

Hand Raking

Hand raking should not be done unless absolutely necessary. The most uniform surface texture can be obtained by keeping the hand work behind the screed to a minimum. This is especially true with crushed stone mixtures. If the operations ahead of the paver are properly performed, if the equipment is in good condition and properly adjusted, and if the paver is not placing the mix at an excessive rate of speed, there should be little or no need for back work. The raker, however, should be alert to a crooked edge on the mat so he can straighten it immediately. He does this by either removing and discarding the mix that bows outside the edge line or by adding mix from the hopper if the edge of the mat is indented. He will also occasionally need to work along the longitudinal joint. If the paver operator properly follows the guide line, the back work will not be necessary.

Surplus hot-mix should not be cast across the mat surface as this will result in nonuniformity of the surface texture—even after proper compaction is obtained.

Constructing Paving Joints

Longitudinal Joints

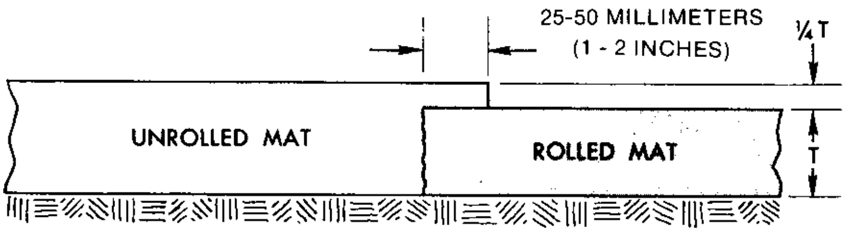
The first lane placed should be true to line and grade, and the longitudinal joint or edge of the paving should have a near vertical face. It is most important that the paving machine keep on a straight, predetermined line of travel. In laying the first lane, the stringline, curb, or other reference line must be used to guide the paver on the proper course. It is also important, for good results, that the thickness adjustment controls on the paver not be over-controlled.

Before the lane is compacted, the material along the unsupported edges should be butted and slightly elevated with a tamping tool or lute. This sets up the edge and permits the full weight of the roller wheel to bear on the material at the extreme edges of the mat.

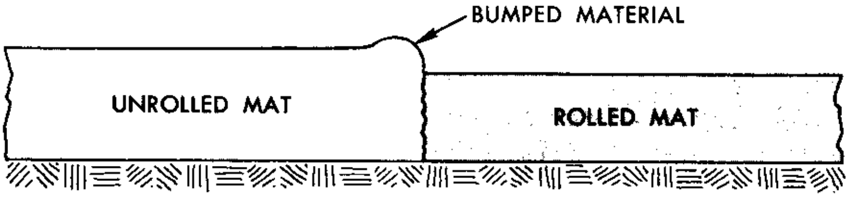
When the adjoining lane is not placed the same day, or if the joint has become distorted by traffic or other means during the day's work, the edge of the lane should be carefully trimmed to line and painted with a very thin coating of emulsified asphalt before the abutting lane is placed.

After the first lane has been placed and rolled, it is important that the adjoining lane be placed so that it uniformly overlaps the first lane by 25 to 50 mm (1 to 2 in.) (Figure 6.23a). The thickness of the overlap should be about one-fourth the desired compacted thickness. The coarse aggregate in the material overlapping the cold joint should be carefully removed and wasted. If another layer is to be placed over the course being spread, the excess coarse aggregate at the joint may be distributed over the unrolled lane with a broom or lute. The overlapping material is then bumped or crowded back onto the hot lane so that the roller can crowd the small excess into the hot side of the joint (Figure 6.23b). When the overlap is excessive, the excess material should be trimmed off so that the bumped ridge of material along the joint is uniform.

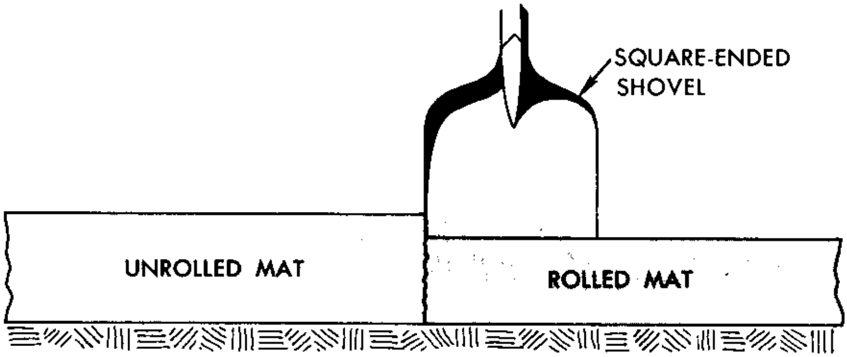
A trimmed joint is sometimes used (Figure 6.23c). This is constructed by removing all freshly placed material that has overlapped the rolled lane. This is best done by trimming the joint immediately behind the paver with a square-ended shovel. In this way the operator can tell where the edge of the cold joint is and gauge his cutoff line accordingly.



(a) OVERLAP OF ADJOINING LANE.



(b) OVERLAP CROWDED BACK READY TO BE ROLLED.



(c) MAKING A TRIMMED JOINT.

Figure 6.23 Constructing and Preparing Longitudinal Joints.

If the lanes are placed simultaneously with two pavers moving in echelon, the loose depths of the mats should match exactly, with no overlap for a hot joint. The joints of a freshly paved mat are usually compacted before the rest of the paved width.

Transverse Joints

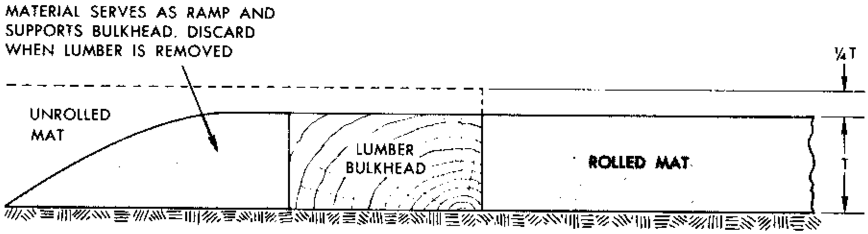
Transverse joints in both binder and surface courses should be carefully constructed and thoroughly compacted to provide a smooth riding surface. Joints should be straightedged or stringlined to assure smoothness and true alignment.

Success in forming a good transverse joint begins with the end of the day's paving operation. In constructing a vertical face transverse joint there are two methods: one uses a bulkhead to form a vertical face, the other has the vertical face sawed into the mat (Figure 6.24).

After the last load of asphalt mix has been placed in the hopper and the material around the spread screws falls below normal, the paver is moved off, and a lumber bulkhead, having the same thickness as the compacted mat, is inserted into a space manually shoveled from the mat (Figure 6.24). The mat is then rolled. The bulkhead and the supporting material beyond the bulkhead are removed before the paver is repositioned the following day.

In the method using the sawed vertical face, the paver operates until the material around the spread screws beings to fall below normal operating level. The screed position is noted and the paver removed. After the material has been shaped for a ramp and compacted, the mat must be sawed before paving operations begin again. The location of the sawed face should be at least 25 mm (1 in.) behind the point of tangency so that a true surface is available at the start. Material ahead of the sawed face is, of course, removed. Placing heavy paper underneath the ramped section will make removal easier when paving operations are resumed.

In every case, the transverse joint should be sprayed or painted with a thin coating of asphalt before fresh material is placed against it. The paver screed should be well-heated before spreading operations are begun. If the fresh material is flush or nearly flush with the older pavement, it will end up too low after rolling has been completed. If this is the case, the fresh material should be loosened with rakes to a depth of about 13 mm (½ in.), and the desired amount of material added and luted smooth. The joint should be rolled both transversely and longitudinally. If the new material is too high after the pass with the tandem roller, it should be loosened with rakes and the excess removed and wasted before continuing the normal rolling operation.


MATERIAL SERVES AS RAMP AND SUPPORTS BULKHEAD. DISCARD WHEN LUMBER IS REMOVED
¼T
UNROLLED MAT
LUMBER BULKHEAD
ROLLED MAT
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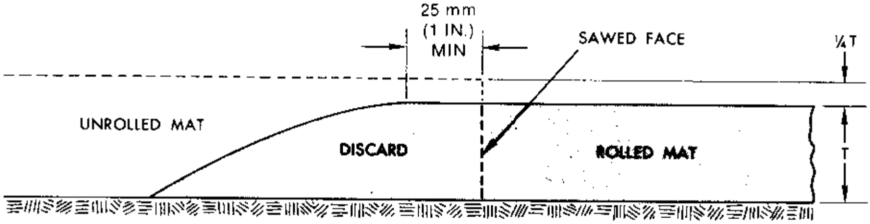
(a) TRANSVERSE JOINT CONSTRUCTED USING A BULKHEAD.



(b) TRANSVERSE JOINT HAVING A SAWED VERTICAL FACE.

Figure 6.24 Constructing and Preparing Transverse Joints.

6.4 Supplemental Roadway Paving and Appurtenances

Introduction

Hand-Spreading Operations

The increasing use of asphalt mixes in construction of roadways has resulted in an increased number of related projects in which asphalt mixes are used. For example asphalt paving mixes are used for constructing driveways, parking areas, shoulders, and sidewalks. Such incidental construction is becoming more and more a part of the paving contract.

Normally, mix requirements for these jobs are the same as for roadway mixes. However, where it is anticipated that the major portion of placing will be by hand, mixes should be designed for good workability. This is easily achieved by decreasing the amount of coarse aggregate in the mix. Placing and compaction methods for hand-placed asphalt mixes are the same as for machine-spread paving.

Certain differences in construction methods are required, especially where special equipment such as sidewalk pavers (Section 6.1) is not available. In addition, there are apt to be places on a regular road-paving job where spreading with a paver is either impractical or impossible. In these cases, hand spreading may be permitted.

Placing and spreading by hand should be done very carefully, and the material distributed uniformly to avoid segregation of the coarse aggregate. When the asphalt mix is dumped in piles, it should be placed far enough ahead of the shovelers to eliminate their moving the entire pile. Sufficient space should be available so that workmen are not required to stand in the material they are working.

Material should not be broadcast or spread from shovels since this causes segregation. Rather, it should be deposited from shovels into small piles and spread with lutes. Any part of the mix that has formed into lumps and does not break down easily should be discarded. After the material has been placed, and before rolling is started, the surface should be checked with templates and straightedges, and all irregularities corrected.

On cool days careful attention must be paid to the mix temperature. Rolling should begin as soon as possible to assure that adequate compaction is achieved.

Supplemental Operations and Appurtenances

Some items, such as roadway widening, shoulders, and superelevations on curves, are considered supplemental paving operations. Handling these projects is described to the extent necessary to achieve quality work. In some cases, such as roadway widening and shoulder construction, there are specially designed equipment and attachments that allow the work to be done easily and efficiently.

Certain roadway appurtenances, such as asphalt curbs, dikes, ditches, spillways, and slopes are increasingly becoming part of the paving contract. For these jobs, special paving equipment, or special asphalt mixtures (or both) may be required.

Roadway Widening

General

Two-lane pavements today should be 7.3 to 7.9 m (24 to 28 ft) wide. Many thousands of miles of primary routes, paved during the 1920s and 1930s, are now inadequate in width and thickness. Narrow roads should be widened before they are strengthened with overlays. Skillful planning can greatly improve the original alignment. One technique is to place most or all widening at one side or the other around curves. Moreover, sharp curves, widened on the inner side to 9.1 m (30 ft) and superelevated, often correct dangerous roadways.

Widening may range from about a metre (a few feet) on one side of a pavement to the addition of full-width traffic lanes on both sides. If warranted by traffic, acceleration and deceleration lanes can be added at intersections.

Construction

Unless proper density can be obtained during construction, widening should be done a year in advance of resurfacing in order to get additional consolidation under traffic.

Traffic movement must be given every consideration throughout construction. Work on both sides of the pavement at the same time and excessive lengths of open trench should not be allowed. Only enough open trench to allow an efficient operation should be permitted.

Asphalt and portland cement pavements are widened by using essentially the same procedures. Both require trenching to adequate width and depth. Cuts are usually made by templates or bits attached to a motor grader or other machine so that the walls and base are neat and true to line and grade. The subgrade of the trench is compacted to the required density or at least rolled until the bottom is firm and free of loose material. Pavement edges are cleaned of all dirt and foreign material.

The edge of the existing pavement is painted or sprayed with an ample quantity of emulsified asphalt. This is where a crack is likely to be reflected up to the finished surface of the overlay, and it is also where most of the surface runoff water flows.

The asphalt base is placed in layers and compacted to the required density (Figure 6.25). Special trench rollers are available for rolling placed in a trench (Figure 6.26). Steel-wheeled vibratory rollers are also good for this type of construction.

Shoulders

General

An asphalt paved shoulder is an excellent safety feature. It also provides high lateral support that enhances the strength of the pavement structure. Asphalt paved shoulders also prevent water from seeping under the travel lanes of the roadway.

A suggested shoulder design section is shown in Figure 6.27.

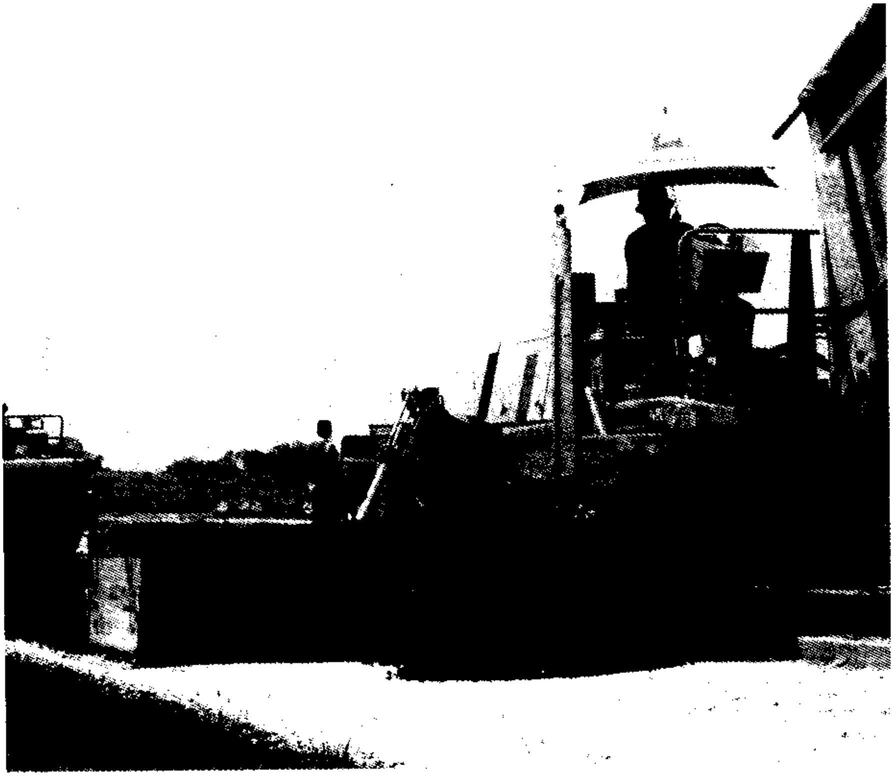


Figure 6.25 Asphalt Concrete Base Applied With Special Paver Attachment in Widening of Road.

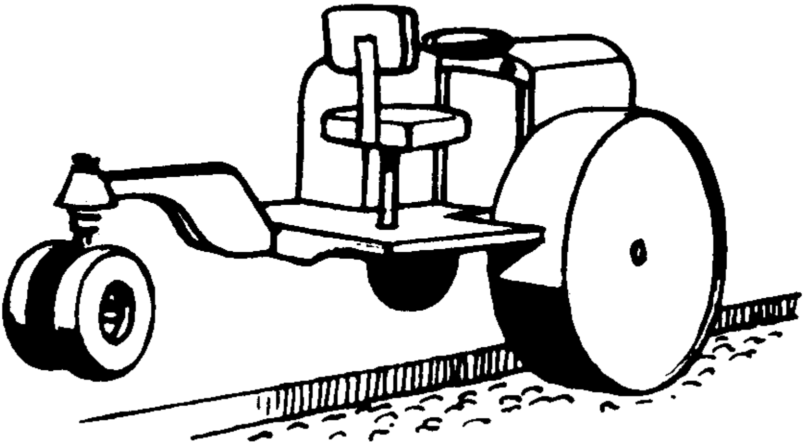


Figure 6.26 Trench Roller.

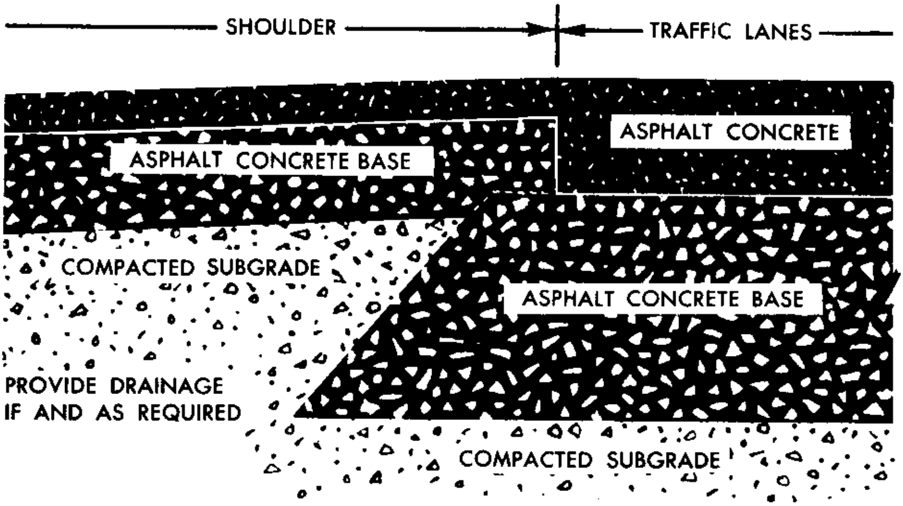


Figure 6.27 Suggested Shoulder Section; Full-Depth Asphalt Pavement.

At one time it was believed that shoulders should be of material different in texture from the pavement proper, to prevent if from being used as an additional traffic lane. Experience has shown, however, that it is best to carry the same surface material across the shoulder. A solid white painted stripe along the edge of the pavement proper affords sufficient demarcation of the shoulder.

After a few months of weathering under different traffic intensity, a color difference occurs that is very effective in ensuring proper traffic use. If rumble is desired, a surface treatment can be placed on the shoulder area. If further contrast is desired, it is easily obtained by a surface treatment on the shoulder portion, using colored aggregate.

Where heavy traffic is expected to use a shoulder, adequate thickness designs should be employed to prevent premature failures.

Construction

Shoulder construction is not necessarily limited to asphalt concrete. The shoulder is sometimes useful as a narrow lane designed to handle light or limited traffic and located adjacent to heavier traveled traffic lanes. Construction procedures, therefore, are similar to regular roadway paving procedures.

Where paved shoulders are included as part of the initial construction and paving of the roadway, the same precautions and procedures for paving the travel lanes are followed. It is important to remember that in laying a surface mix adjacent to any finished surface area, the loose mix must be placed thick enough so that when compacted the finished surface is at the desired grade.

Where shoulders are added to an existing paved roadway, the same procedures and precautions stated for pavement widening are observed. It is important that the new shoulder aids rather than retards surface-water runoff. Also, the vertical face of the existing pavement should be given an adequate tack coat of asphalt because cracking is most likely to occur between these two surfaces and admit water beneath the pavement.

Superelevation

General

Paving superelevated curve sections on new construction does not necessarily pose unique problems for a paving crew. Prepared foundation material is usually superelevated also. There is a stretch of roadway where the transverse slope of the pavement surface on the straight portion (tangent) goes through a transition to the superelevation slope.

Many existing older roads that require widening and resurfacing are not properly superelevated in the curve sections. In order to safely accommodate faster traffic, it is necessary to build up superelevated curves with asphalt concrete.

Initial Construction Methods

For new construction, where the paving is placed directly on the prepared subgrade, the foundation is sloped accordingly so that the pavement has a uniform thickness on the tangent. Manually operated screed controls should present no particular problems. As the paver proceeds through the transition to the superelevated section, the screed tilts accordingly, and little if any adjustment in the screed controls is required.

The transverse slope of pavers using automatic screed controls is maintained by a pendulum and a dial setting for the desired slope. Sensors riding an erected or traveling stringline maintain grade control for the end of the screed nearest the sensor. The depth at the other end is dependent on the transverse slope control or dial setting. When the paver reaches the point where the slope begins to increase (or decrease) to (or from) the superelevated slope, the change in slope is slowly dialed in as the paver proceeds through the transition. Stakes spaced along the transition having the required slopes at these points can be used as a guide. The change in slope is gradual and small errors in dialing do not affect the riding quality of the pavement.

Building Up Superelevation

In building up the superelevation of curves with asphalt mixes, wedges are constructed in lifts which taper from 50 mm (2 in.) in thickness on the outside edge to 25 mm (1 in.) in thickness on the inside edge. Successive layers also increase in width. For instance, Figure 6.28 shows a 150 mm (6 in.) lift made up of three successive layers of 2.4, 3.0 and 3.7 m (8, 10, and 12 ft) in width for a 3.0 m (10 ft) lane. This does not increase the superelevation of the inside lane. Superelevation of curves can be accomplished either with pavers or motor graders.

Asphalt Curbs and Dikes

General

Asphalt curbs are popular as accessories to new road and street paving. On dual-lane turnpikes and interstate highways, asphalt dikes (curbs) are often constructed at the outer edge of the paved shoulders on elevated grades to prevent slope erosion.

Asphalt curbs and dikes are constructed with automatic curbing machines (Figure 6.29). These machines lay, compact, and finish straight or curved curbs for streets, traffic islands, dikes, raised medians, and parking areas. No forms are necessary. Various interchangeable mold patterns provide a wide selection of curbing shapes.

Hot-mix asphalt is placed in the open hopper of the curbing machine. A motorized worm gear, or screw, pushes the mixture out through the mold form under pressure. This pressure provides compaction and also causes the machine to move forward about 1.22 to 2.3 m/min (4 to 7½ fpm).

Mix Requirements

Basically, a well-graded asphalt-concrete surface course mixture is very similar to the type required for asphalt curbs. Asphalt curb mixes differ in that the quantity and grade of asphalt cement are highly important factors in obtaining satisfactory curb and gutter mixes.

The quantity of asphalt, regardless of the grade used, should be 0.5-1.0 percent more than that normally used for highway paving mixtures of the same aggregate gradation. This additional asphalt increases durability and facilitates compaction.

Usually curb and gutter mixes perform better with AC-20 (AR-8000 or 60-70 penetration) or harder grade asphalt cement than with softer asphalt.

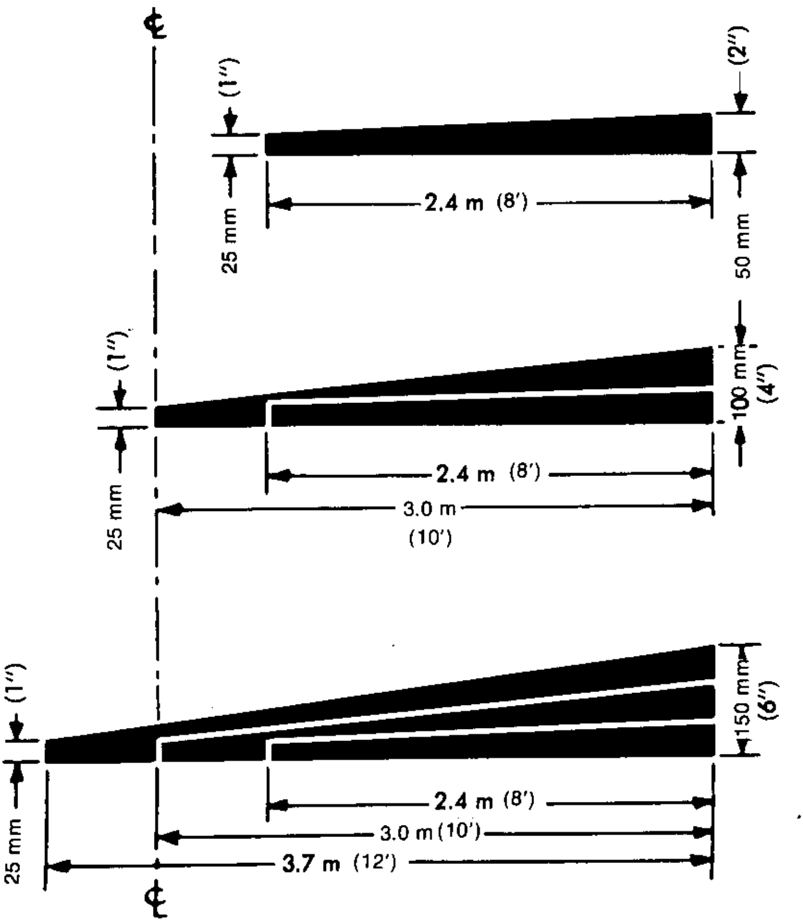


Figure 6.28 Building Up Superelevated Curves With Asphalt Mix.

Therefore, when a softer asphalt is being used, a practical solution for small quantities of curb mix is to add a little extra filler to the mixture.

It is essential that backfill material be placed behind an asphalt curb when vehicle tires are to rest against the curbing. Otherwise, the tires may cause excessive indentation or displacement to the curbing.

Construction

It is the usual practice to place string or chalk guidelines over the foundation that, in most cases, is the existing pavement. The outside edge of the curb or dike should be at least 50 mm (2 in.) in from the outside edge of the pavement on which it is placed. A thin coating of emulsified asphalt or rapid-curing asphalt is applied to the pavement surface to ensure that the curbing mixture is bonded to the pavement.

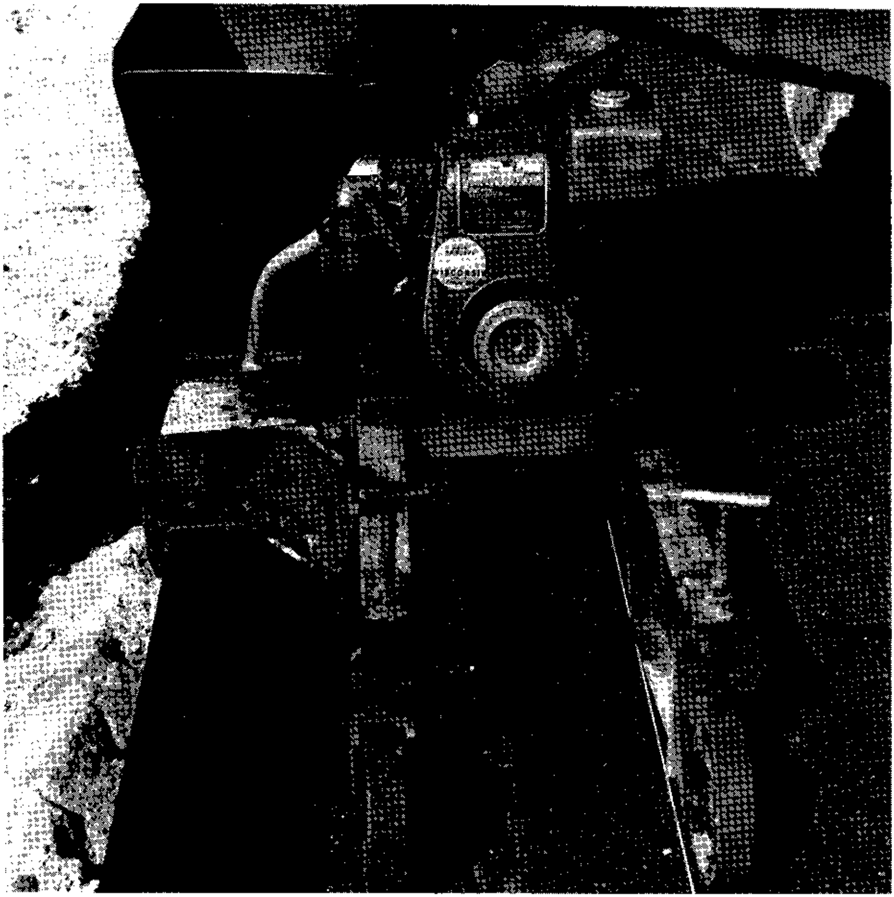
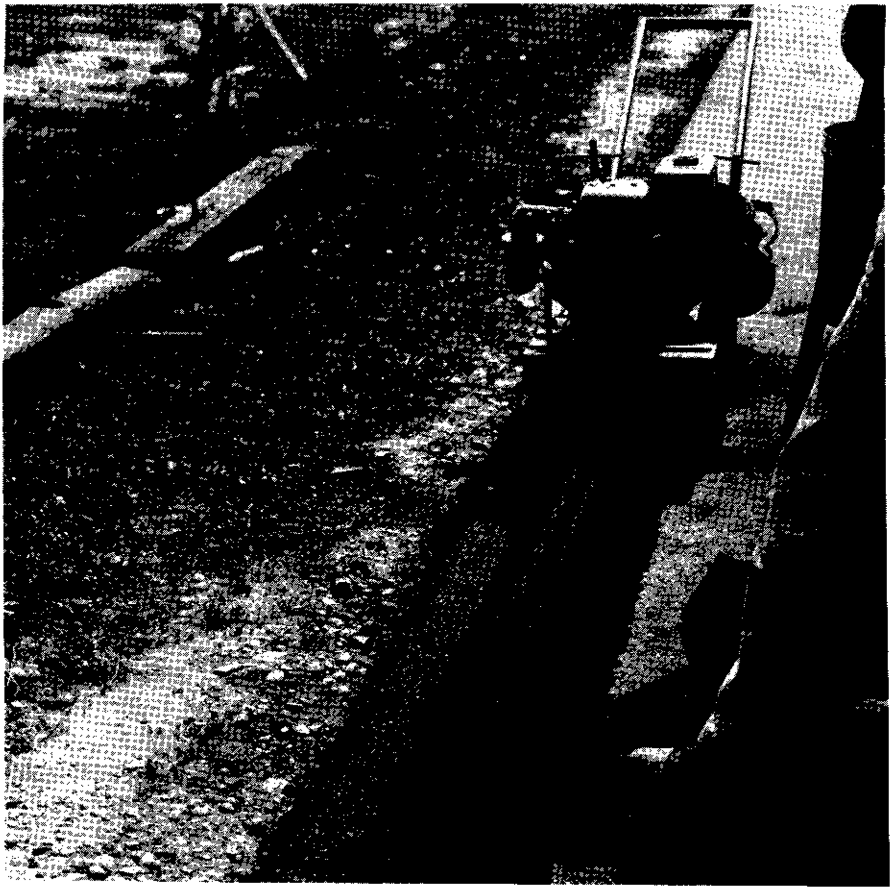


Figure 6.29 Two Types of Curbing Machines. Interchangeable Mold Patterns Provide a Wide Variety of Curbing Shapes.

The temperature of the mix at the time of placement is very important. If the mix temperature is too low, it is not possible to obtain adequate compaction. If the mix temperature is too high, the curb may slough off. Normally a temperature range of 120 to 140° C (250 to 285° F) is satisfactory.

Machine-laid work usually does not require additional compaction. But in areas where it is evident that compaction is inadequate, the mix should be adjusted, forward movement of the machine retarded by braking, or other measures taken to provide adequate compaction.



Asphalt Ditches and Spillways

General

Ditches, as differentiated from gutters, occur parallel to the rural highway at the foot of the outer embankment, or between dual-lane pavements having a depressed median strip. They may be so wide and gentle in slope that grass sod is sufficient. In rolling terrain, however, there are many locations where pavement is required to prevent costly erosion. In cut sections, it is often desirable to construct ditches over the tops of slopes and around the ends to prevent slope erosion and heavy slides. Spillways down the embankments to carry off water collected by curbs or dikes are subject to high water velocities. In all cases, such channels should be sufficient so that the heaviest anticipated runoff is carried away smoothly.

In some instances, very large outfall ditches may be necessary to carry runoff completely away from the right-of-way. The type of paving used depends upon the gradient and type of soil. In areas of very erosive soils careful design is of great importance.

Mix Requirements

Asphalt concrete makes an excellent ditch or spillway lining material. Highway surface course mixes generally need only be modified by increasing the asphalt and mineral filler contents as necessary to ensure watertightness and durability. The resulting lower stability is not important but actually adds to the capacity of the lining to adjust to minor settlement without cracking.

The use of a permeable yet erosion-resistant lining for ditches is sometimes desirable. The permeability of such a lining permits water to escape from behind the lining, preventing any damage from hydrostatic pressure and permitting damage of adjacent areas. It affords a tough, stable, erosion-resistant surface that is permeable because of the aggregate grading used.

Construction

The subgrade is thoroughly cleaned of all organic and loose material. If conditions are favorable for growth of weeds and other plants in the subgrade, it is necessary to treat the soil with a soil sterilant prior to placing the asphalt lining.

The soil is compacted sufficiently to attain stability, particularly on the side slopes. It is also trimmed as necessary to make it smooth and uniform.

Linings are made as thick as necessary for the use and stresses involved, but practical experience indicates that a finished thickness of 40 mm (1½ in.) of asphalt concrete should be the minimum thickness specified.

There are special slip-form paving machines for placing at least part of the ditch linings. Spreader boxes, paving machines, and even hand-placing procedures may be used.

Compaction of asphalt mixes in ditches and spillways is accomplished by hand tamps, weighted slip forms, small shoe type vibratory compactors, or small rollers controlled by cables.

Asphalt Paved Slopes and Revetments

General

Slope paving may be performed on both cuts and embankments. The pavement is placed on the face of a cut slope, for some distance above the ditch, to prevent undercutting and consequent slides. This paving is an extension of the outer side of the ditch itself, although it may have a different thickness. The essential item, in addition to a dense composition and high asphalt content, is firm anchorage to prevent the intrusion of water.

Embankment slopes are paved to prevent erosion. One type of embankment paving is that used under the ends of open-type bridges. Another form is levee slope paving. An open-type hot mix used for this form of paving provides erosion control and permits moisture from the embankment to escape.

Revetments are paved embankment slopes adjacent to moving water. In most instances such pavements are used to prevent scouring and undermining. With torrential streams, or adjacent to lakes or oceans where wave action is a factor, substantial anchorage of the revetments may be required. Normally, dense-graded asphalt mixes with relatively high asphalt content are required.

Mix Requirements

Asphalt concrete such as described for ditch paving is normally recommended. Again, stability of the mix is not the important consideration, but rather the durability and watertightness where necessary. The use of a higher asphalt content and such grades as AC-20 or 60-70 pen is recommended.

Open-graded asphalt mixes permit the relief of hydrostatic pressure in the embankment. Even so, a relatively high asphalt content is recommended.

Construction

Construction procedures for smaller jobs will probably consist mostly of hand-spreading methods or material placed by a towed-type paver raised or lowered on the slope with cables. For larger paved areas, a paving machine may be raised and lowered on the slope with cables. Special slip-form machines devised for the job are sometimes used.

Rolling asphalt mix will in most every instance require a steel-wheeled or vibratory roller raised and lowered on the slope with cables if adequate compaction is to be obtained.

Chapter 7

Compacting Hot-Mix Asphalt

Compaction is the process of compressing asphalt hot-mix into a smaller, denser volume. It is accomplished by consolidating the asphalt coated mix and increasing the density (weight to volume ratio) of the mixture. Compaction is considered successful when the finished mat reaches optimum void content and density.

Compaction is the final stage of hot-mix asphalt construction. It is the stage at which the full strength of the mixture is developed and the smoothness and texture of the mat are established. Consequently, during the compaction process, those involved must be particularly observant and knowledgeable.

This chapter will discuss the principles of the compaction process and the standard compaction or rolling equipment that is utilized. Also discussed are the various properties; texture, grade and density, that are to be achieved in the final compacted pavement.

SECTION 7.1 Compaction Basics
- ☐ Need for Compaction
- ☐ Fundamental Principles
- ☐ Factors Affecting Compaction

SECTON 7.2 Compaction Equipment

SECTION 7.3 Rolling Procedures
- ☐ Introduction
- ☐ Transverse Joints
- ☐ Longitudinal Joints
- ☐ Breakdown Rolling
- ☐ Intermediate Rolling
- ☐ Finish Rolling

SECTION 7.4 Checking the Finished Pavement
- ☐ Surface Texture
- ☐ Surface Tolerance
- ☐ Pavement Density
- ☐ Additional Procedures

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VISUAL AIDS

1. *Rolling and Compaction of Asphalt Pavement, (VA-21),* slide show (40 min.), Asphalt Institute.

7.1 Compaction Basics

Need for Compaction

During compaction of asphalt hot mix three very important factors that will affect pavement performance take place. The asphalt coated aggregate particles are pressed together, air voids are reduced, and mix density (weight to volume ratio) increases.

1) This squeezing together of the aggregates increases their surface to surface contact and interparticle friction, resulting in higher mix stability and pavement strength.

2) The reduction of air voids to the optimum level in the mix produces a pavement that is nearly impermeable. In an undercompacted mix the voids tend to be interconnected thereby permitting the intrusion of air and water into the pavement structure. The air oxidizes the asphalt binder, making it brittle and causing the mix to crack under the repeated deflections due to traffic loads. Water permeating into and through the pavement can lead to stripping of the asphalt from the aggregate, weakening of the base and subgrade soils under the pavement, or freeze-thaw damage in colder climates. Premature pavement distress can occur in any of these situations.

3) Finally, if a high level of compaction (concurrently with items 1 and 2 above) is not achieved during construction, subsequent traffic will further consolidate the mix. This will occur principally in the wheelpaths and the resulting ruts could be a safety hazard to traffic using the road.

Fundamental Principles

The goal of compacting an asphalt concrete pavement is to achieve the optimum air void content and provide a smooth riding surface. Behind the paver, the mat has 15 to 20% air voids. It is the task of the rollers to reduce that void content to 8% or less for dense-graded mixes. At that level, the voids are usually not interconnected and the detrimental effects of air and water (primarily ravelling and stripping) are avoided. Caution must be exercised, however, not to compact the pavement to a very low air void level (less than 2%). This minimum void level is necessary to allow for thermal expansion without causing flushing and mix instability. Figure 7.1 is a graphic representation of the effect of air voids on pavement durability.

Even with a properly designed mix, compaction of the mat is possible only if two conditions exist: correct mix temperature and mix confinement. At temperatures between 85° C (185° F) and 150° C (300° F), asphalt is fluid enough to act as a lubricant in the compaction process aiding movement of the mix aggregates into a dense configuration. As it cools below this temperature range, asphalt becomes very stiff and further mix densification is extremely difficult. Therefore, compaction must be completed before the

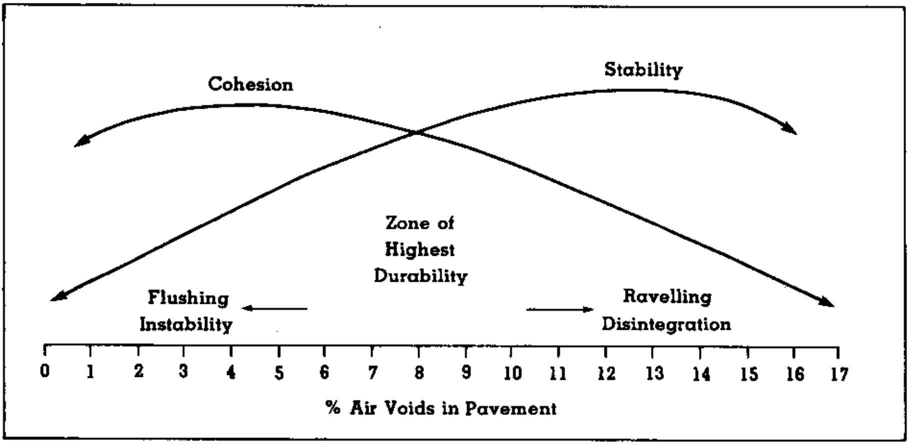


Figure 7.1 Pavement Durability vs. Air Voids.

mix cools to 85° C (185° F). Initial mix temperature, air temperature, wind, humidity, mat thickness, and temperature of the surface being paved affect the rate of cooling and thus the time available for compaction. Figure 7.2 shows a typical cooling rate curve for various mix temperatures.

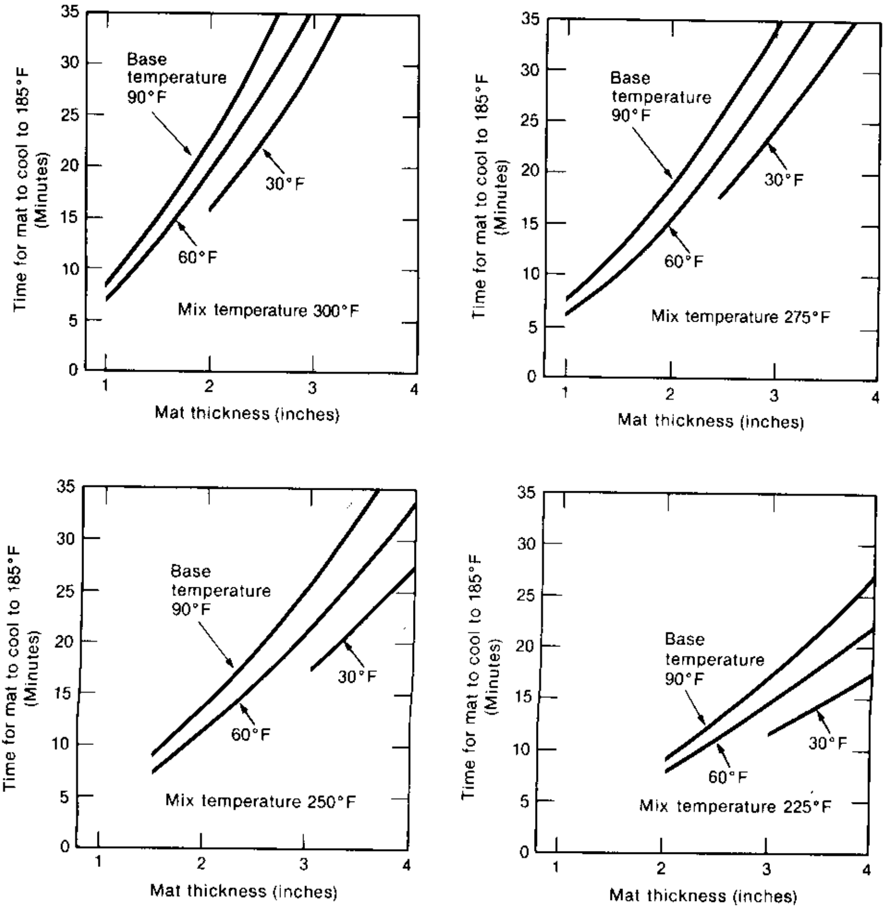
Even when the mat is at the correct temperature, density can only be obtained when the mix being compacted is adequately confined. This confinement consists of three forces: the compressive force of the roller, the forces within the mix that resist the rollers, and the supporting forces provided by the stable surface beneath the mat. The compressive force of the rollers comes from either their weight or a combination of weight and dynamic energy due to vibration. Resisting forces in the mix are the combined effect of interparticle friction of aggregates and the asphalt's viscosity. The supporting force of the subgrade results from its stability and firmness. When mix density and temperature reach a point where the resisting forces equal the compressive force of the rollers and the resistant forces of the subgrade, equilibrium is reached and the compaction process is complete. Figure 7.3 illustrates this concept.

Factors Affecting Compaction

Factors affecting compaction can be categorized into three classes: mix properties, environmental conditions, and layer (lift) thickness.

Mix Properties

Properties of various asphalts and aggregates have a pronounced effect on the workability of mixes at different temperatures. These properties, and the temperature of the mix at the time of compaction, must be considered when deciding on a compaction procedure.



Wind velocity—10 knots. Atmospheric temperature—same as base.

Note: "Base Temperature" is the temperature of the surface upon which the asphalt mat is placed.

*185°F is the temperature of the mat measured ¼ to ½ inch below the mat surface. The average temperature of the entire mat thickness when this temperature is reached, is approximately 175°F.

On a subgrade (base temperature) of 30°F, placing of thicknesses less than those shown on the curves is not recommended.

(Conversion: 1 inch = 25mm, °F = 1.8 (°C) - 32)

Figure 7.2 Time Allowed for Compaction, Based on Temperature and Thickness of Mat and Temperature of Underlying Base.

(a) *Aggregate*

Gradation, surface texture and angularity are the primary aggregate characteristics that affect workability of the mix. As the maximum aggregate size or percentage of coarse aggregate in the mix increases, the workability decreases and greater compactive effort is required to achieve target density. Similarly, a rough surface texture, as opposed to a smooth, glassy aggregate surface, results in a more stable mixture and requires greater compactive effort. Mixtures that are produced from gravels

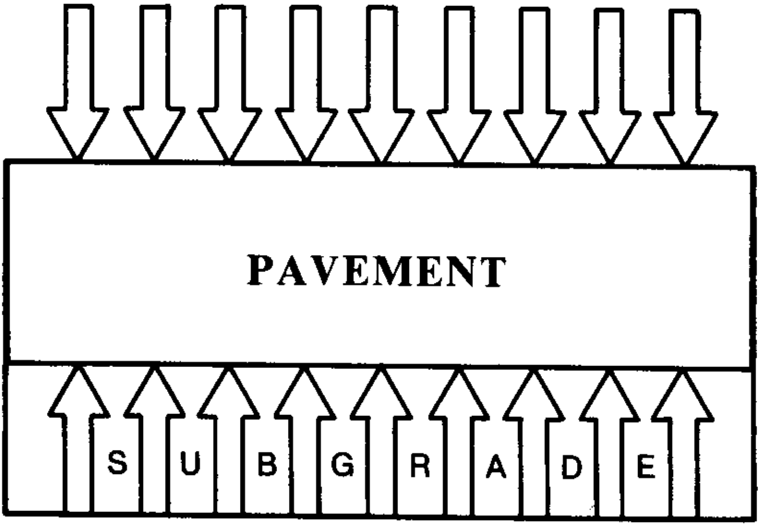


Figure 7.3 Forces at Work During Compaction.

frequently are more rounded than quarry rock and thus more workable and easier to compact.

Natural sands are often added to mixes in the interests of economy and to improve workability. Too much sand, particularly in the middle particle sizes—around the 0.60mm (No. 30) mesh sieve—will result in tender mixes (mixes with high workability, but low stability). Tender mixes are easily overstressed by heavy rollers and too much rolling. They are often susceptible to scuffing and displacement by traffic after several weeks in place.

The fines or filler content in the mix will also affect the compaction process. It is the combination of filler and asphalt that provides the binding force in asphalt hot-mix pavements; therefore, the mix should contain sufficient fines to combine with the asphalt to produce the necessary cohesion when the mix cools. The addition of mineral filler will help to offset the tenderness or slow-setting properties of mixes containing too much sand. Conversely, if a mix contains too many fines it will become "gummy" and very difficult to compact.

(b) *Asphalt*

At room temperatures asphalt is virtually a solid whereas at 121° - 149° C (250° - 300° F) it is a fluid. As previously mentioned, for a mix to be properly compacted, the asphalt in it must be fluid enough to permit the aggregate particles to move past one another. In effect then, the asphalt acts as a lubricant during compaction. As the mix cools, the asphalt loses fluidity (becomes more viscous). At temperatures below 85° C (185° F), the asphalt, in combination with the fines in the mix, begins to bind the aggregate particles firmly in place. Consequently, compaction of the mix is extremely difficult once the mix has cooled to 85° C (185° F).

The grade of asphalt that is used and the temperature at which the mix is produced determine its viscosity. Other factors being equal, a higher viscosity asphalt in the mix may require a slightly higher compaction temperature and/or greater compactive effort.

The asphalt quantity in the mix will also affect workability. As the asphalt content increases, the film thickness of the asphalt on the aggregate particles increases. At compaction temperatures, this increased film thickness increases the lubricating effect of the asphalt and up to a certain point makes compaction easier.

(c) *Mix Temperature*

The temperature at which an asphalt mixture is produced affects both the ease of compaction and the time it takes for the mix to cool to 85° C (185° F). Up to a certain point the hotter the mix, the more fluid the asphalt and the less resistant the mix is to compaction. The upper limit for mix temperature is approximately 150° C (300° F). Higher temperatures may result in damage to the asphalt by accelerated hardening. Within these limiting values 85° C to 150° C (185° F to 300° F), the best temperature to begin rolling (compaction) is the maximum temperature at which the mix will support the roller without moving horizontally.

At the time of placement, the mix temperature is uniform throughout the thickness of the mat. However, the top and bottom surfaces cool more rapidly than the interior because they are in contact with the cooler air and subgrade or underlying pavement.

Environmental Effects

As illustrated in Figure 7.2, the rate at which the mix cools affects the length of time during which density can and must be achieved. Ambient (air) temperature, humidity, wind and the temperature of the surface under the mix affect the rate of cooling. Cool air temperatures, high humidity, strong winds, and cool surfaces alone or together shorten the time in which compaction must take place. This may make compaction more difficult.

Layer Thickness

Generally speaking, it is easier to achieve target density in thicker layer (lifts) of asphalt concrete than in thinner ones. This is because the thicker the mat the longer it retains its heat and the longer the time during which compaction can be achieved. This can be used to advantage when rolling lifts of highly stable mixes that are difficult to compact, or when paving in weather that can cause rapid cooling of thin mats.

Alternatively, increased course thickness can permit lower mix temperatures to be used because of the reduced rate of cooling.

Figure 7.4 gives a summary of items influencing compaction.

ITEM	EFFECT	CORRECTIONS*
Aggregate		
• Smooth Surfaced	Low interparticle friction	Use light rollers Lower mix temperature
• Rough Surfaced	High interparticle friction	Use heavy rollers
• Unsound	Breaks under steel-wheeled rollers	Use sound aggregate Use pneumatic rollers
• Absorptive	Dries mix—difficult to compact	Increase asphalt in mix
Asphalt		
• Viscosity		
—High	Particle movement restricted	Use heavy rollers Increase temperature
—Low	Particles move easily during compaction	Use light rollers Decrease temperature
• Quantity		
—High	Unstable & plastic under roller	Decrease asphalt in mix
—Low	Reduced lubrication—difficult compaction	Increase asphalt in mix Use heavy rollers
Mix		
• Excess Coarse Aggregate	Harsh mix—difficult to compact	Use heavy rollers
• Oversanded	Too workable—difficult to compact	Reduce sand in mix Use light rollers
• Too Much Filler	Stiffens mix—difficult to compact	Reduce filler in mix Use heavy rollers
• Too Little Filler	Low cohesion—mix may come apart	Increase filler in mix
Mix temperature		
• High	Difficult to compact—mix lacks cohesion	Decrease mixing temperature
• Low	Difficult to compact—mix too stiff	Increase mixing temperature
Course Thickness		
• Thick Lifts	Hold heat—more time to compact	Roll normally
• Thin lifts	Lose heat—less time to compact	Roll before mix cools Increase mix temperature
Weather Conditions		
• Low Air Temperature	Cools mix rapidly	Roll before mix cools
• Low Surface Temperature	Cools mix rapidly	Increase mix temperature
• Wind	Cools mix—crusts surface	Increase lift thickness

*Corrections may be made on a trial basis at the plant or job site. Additional remedies may be derived from changes in mix design.

Figure 7.4 Summary of Influences on Compaction.

7.2 Compaction Equipment

General

Compaction is done by any of several types of compactors, or rollers—vehicles which, by their weight or by exertion of dynamic force, compact the pavement mat by driving over it in a specific pattern.

Self-propelled rollers are normally required for the compaction of asphalt concrete mixtures. Towed-type rollers should not be used, but hand-held rollers or vibrating plates can be used in small areas inaccessible to larger rollers. Typical self-propelled compaction rollers include these three types:

- Static steel-wheeled.
- Pneumatic-tired.
- Vibratory.

Before any roller is used on a project it should be inspected to see that it is in good mechanical condition. Where applicable, the following items should be checked:

- Total weight of roller.
- Weight per inch of width (steel-wheeled rollers).
- Average ground contact pressure (pneumatic-tired rollers).
- Mechanical condition.
- Precise steering.

Static Steel-Wheeled Rollers

Steel-wheeled rollers by configuration are two-axle tandem rollers (Figure 7.5) or three-wheeled rollers. They vary in weight from 2.7 to 12.7 tonnes (3 to 14 tons) or more. Most have wheels to which ballast can be added to increase the weight.

Steel-wheeled rollers should be checked for wear of the wheel rims. A sharp metal straightedge may be used for this check. The roller should not be used if grooves or pits have worn into the rolling drum. These rollers have scrapers for keeping rollers clean and wetting pads to keep rollers wet so that they do not pick up asphalt during the roller operation. If they are worn excessively they should be replaced.

Pneumatic-Tired Rollers

Self-propelled pneumatic-tired rollers (Figure 7.6) have two to eight wheels in front and four to eight wheels in the rear. The wheels on these rollers generally oscillate (axle moves up and down) and some may wobble. Wobble-wheel rollers, however, should not be used for compacting asphalt concrete. Self-propelled pneumatic-tired rollers vary in weight from 2.7 to 31.8 tonnes (3 to 35 tons). Ballast can be added to the machines to increase the weight.

Several self-propelled pneumatic-tired rollers have a device to change tire inflation while the roller is operating. This "inflation-on-the-run" system automatically maintains any preset tire pressure or can raise or lower the inflation pressure while the roller is operating. Some compacting conditions and requirements require different inflation pressures. However, tire inflation pressures should all be within a few p.s.i. between tires.

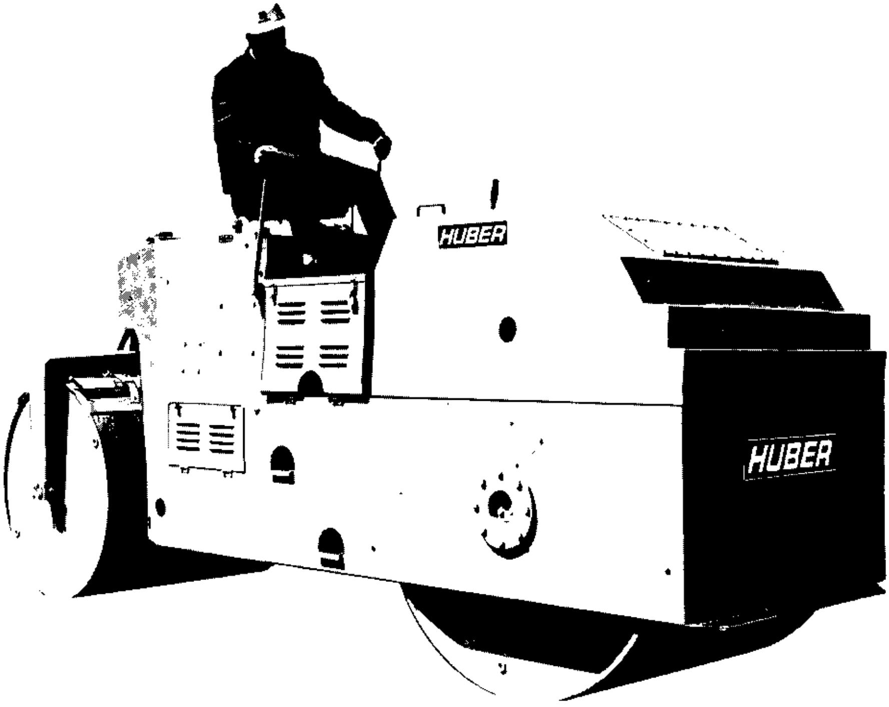


Figure 7.5 Tandem, Static Steel-Wheeled Roller.

Vibratory Rollers

Vibratory rollers are made with one or two smooth-surfaced steel wheels 0.8 to 1.5 m (2.5 to 5 ft) in diameter and 0.9 to 2.7 m (3 to 9 ft) in width (Figure 7.7). They vary in static weight from 1.4 to 15.5 tonnes (1.5 to 17 tons). Vibratory rollers are used for compacting any type of asphalt mixture but should not be used in the vibratory mode when the mat thickness is 37.5 mm (1.5 in) or less. With some, however, it is necessary to adjust the resonance of the dynamic force to the foundation and type of material being compacted.

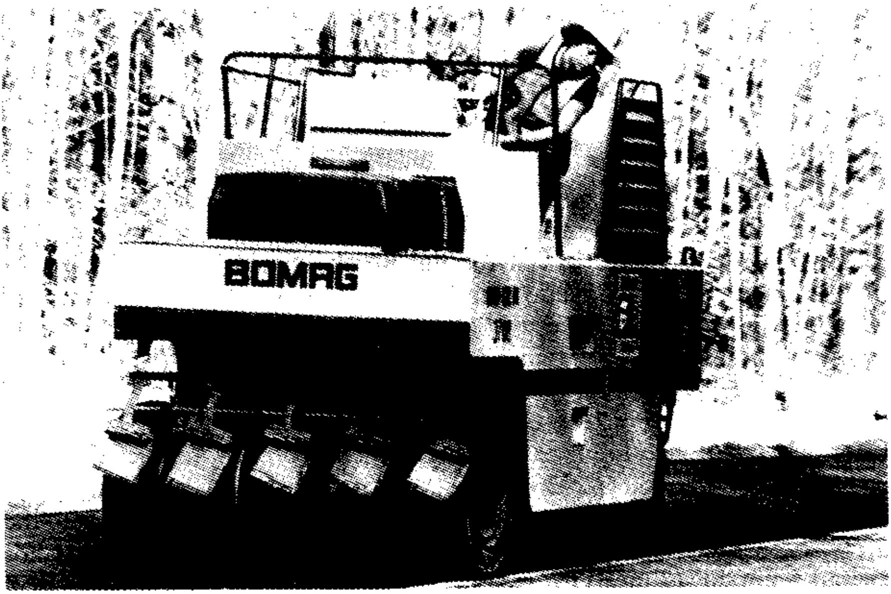


Figure 7.6 Self-Propelled Pneumatic-Tired Roller.

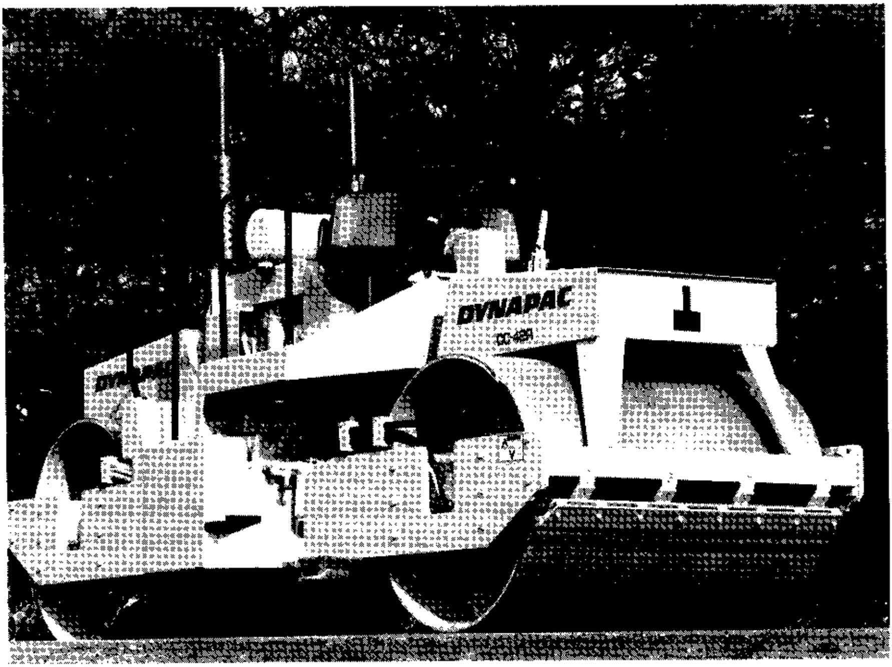


Figure 7.7 Self-Propelled Tandem Vibrating Roller.
(Courtesy Dynapac Mfg. Co.)

7.3 Rolling Procedures

Introduction

Rolling should start as soon as possible after the hot-mix has been spread. Rolling consists of three consecutive phases: breakdown or initial rolling, intermediate rolling, and finish rolling. Breakdown rolling compacts the material beyond the compaction imparted by the paver, to obtain practically all of the needed density. Intermediate rolling densifies and seals the surface. Finish rolling removes roller marks and other blemishes left from previous rolling.

Two or more rollers are needed on most projects other than small jobs such as driveways. During rolling, roller wheels are kept moist with only enough water to avoid picking up material. Do not use fuel oil on roller wheels or pneumatic tires. Rollers move at slow but uniform speed with the drive roll or wheels nearest the direction of travel. The speed should not exceed 5 km/hr. (3 mph) for steel-wheeled static or vibratory rollers or 8 km/hr. (5 mph) for pneumatic-tired rollers. Rollers must be kept in good condition, capable of being reversed without backlash. The line of rolling should not be suddenly changed or the direction of rolling suddenly reversed, thereby displacing the mix. Any pronounced change in direction should be made on stable material.

If rolling causes material displacement, the affected areas are loosened at once with lutes or rakes and restored to their original grade with loose material before being re-rolled. Heavy equipment, including rollers, should not be permitted to stand on the finished surface before it has thoroughly cooled or set.

Rolling freshly placed asphalt mix is done in the following order:

1. Transverse joints,
2. Longitudinal joints (when adjoining a previously placed lane),
3. Initial or breakdown rolling,
4. Second or intermediate rolling, and
5. Finish rolling.

Transverse Joints

When a transverse joint is placed next to an adjoining lane, the first pass is made with a steel-wheeled roller moving along the longitudinal joint for a short distance. The surface is then straightedged and corrections made if necessary. The joint is then rolled transversely, with the roller on the previously laid material, except for a 150 mm (6 in.) projection of the wheels when using a tandem (Figure 7.8) and of one rear wheel when using a three-wheeled roller. This operation should be repeated with successive passes covering 150 to 200 mm (6 to 8 in.) of fresh material until the entire width of a drive roll is on the new mix.

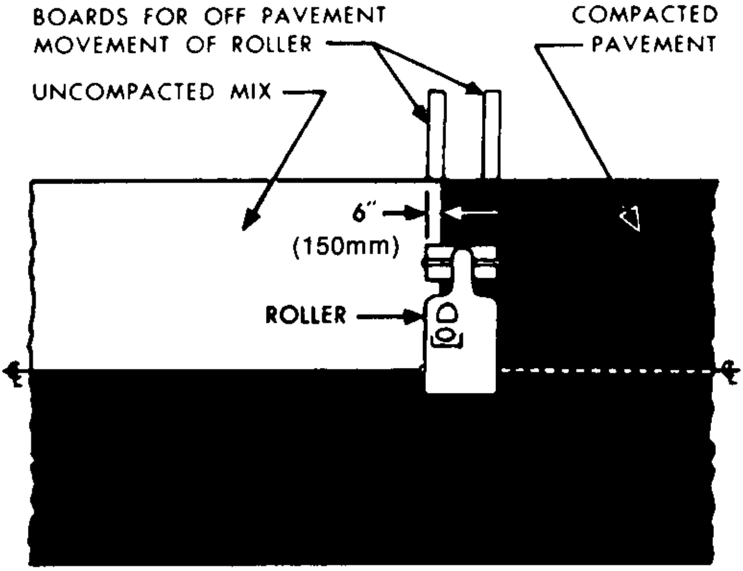


Figure 7.8 Rolling a Transverse Joint.

Boards of proper thickness should be placed at the edge of the pavement to provide for off-the-pavement movement of the roller. If boards are not used, transverse rolling must stop 150 to 200 mm (6 to 8 in.) short of the outside edge in order to prevent damage to the edge. The outside edge then must be rolled out later when rolling longitudinally.

Longitudinal Joints

Longitudinal joints should be rolled directly behind the paving operation. The roller should be shifted to the previously-placed lane so that not more than 150 mm (6 in.) rides on the edge of the newly spread material left by brooming and bumping (Figure 7.9). Here it is operated to pinch and press the uncompacted material into place. The roller should continue to roll along this line, its position being shifted gradually across the joint until a thoroughly compacted, neat joint is obtained. Vibration should not be used when operating on a cold mat.

When rolling a longitudinal joint against a concrete gutter lip, the roller should not span the joint such that the weight of the roller is partially on the gutter. This could result in the roller bridging over a portion of the pavement and thus insufficient compaction. One common way to obtain good compaction is to require that the elevation of the edge of the finished pavement be slightly higher than the adjacent gutter. This ensures that the full compactive force of the roller will be on the asphalt mix.

Breakdown Rolling

Breakdown rolling is best accomplished with steel-wheeled rollers. Vibratory or static-weight tandem rollers are recommended. Pneumatic-

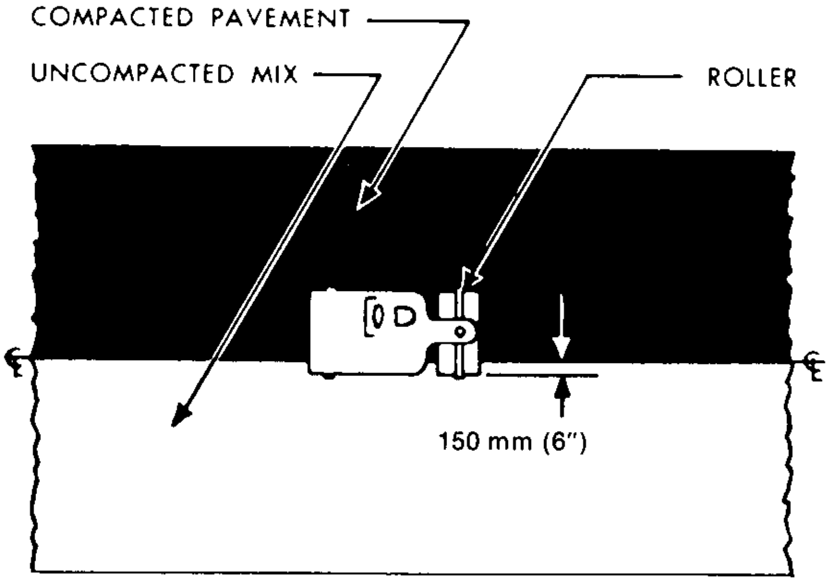


Figure 7.9 Rolling a Longitudinal Joint.

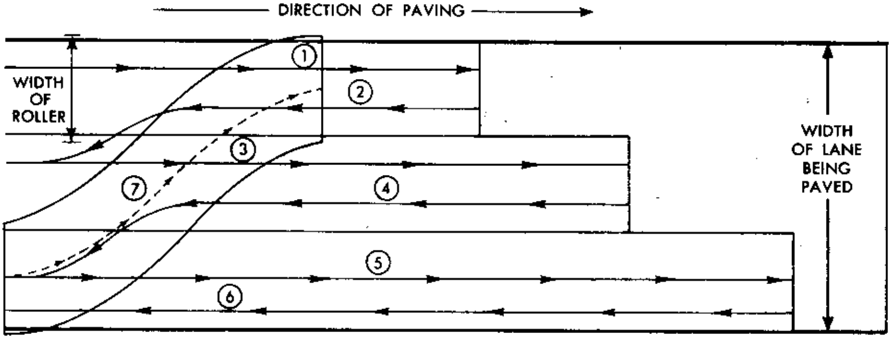
tired rollers, however, have also been used successfully for breakdown rolling, but normally should be limited to base courses.

The weight of the roller used for breakdown rolling depends to a large degree upon the temperature, thickness, and stability of the mix being placed. Generally, a roller weighing from 7.3 to 10.9 tonnes (8 to 12 tons) is used for this operation.

It is important to start the rolling operation on the low side of the spread, which is usually the outside of the lane being paved, and progress toward the high side. The reason for this is that asphalt mixtures, when hot, tend to migrate towards the low side of the spread under the action of the roller. If rolling is started on the high side, this migration is much more pronounced than if rolling progresses from the low side. When adjoining lanes are placed, the same rolling procedure should be followed. This is important to assure proper cross slope and/or superelevations are maintained, but only after compaction of fresh mix at the longitudinal joint with 150 to 200 mm (6 to 8 in.) of roller width.

A rolling pattern that provides the most uniform coverage of the lane being paved should be used. Rollers vary in widths, and a single recommended pattern that applies to all rollers is impractical. For this reason, the best rolling pattern for each roller being used should be worked out and followed to obtain the most uniform compaction across the lane.

The rolling pattern not only includes the number of passes, but also the location of the first pass, the sequence of succeeding passes, and the overlapping between passes. Rolling speed should not exceed about 5 km/hr (3 mph). In addition, sharp turns and quick starts or stops are to be avoided. For thin lifts, a recommended rolling pattern for static steel-wheeled rollers is shown in Figure 7.10. The rolling operation should start from the edge of the spread on the low side with the roller moving forward as close behind the paver as possible. The second movement of the roller should be to reverse in the same path until the roller has reached previously compacted material. At this point it should swing over and move forward along path number 3, again going as close as possible behind the paver. The fourth movement is reversal in the third path and a repetition of the previous operation. After the entire width of the mix being placed has been covered in this fashion, the roller should swing across the spread to the low side and repeat the process. With this pattern, the lap of the roller with succeeding passes need not be more than 75 to 100 mm (3 to 4 in.)



This is a recommended rolling pattern. Every pass of the roller should proceed straight into the compacted mix and return in the same path. After the required passes are completed, the roller should move to the outside of the pavement on cooled material and repeat the procedure.

Figure 7.10 Correct Rolling Pattern.

For thick-lift construction, the rolling process should start 300 to 375 mm (12 to 15 in.) from the lower unsupported edge until the center portion of the spread is compacted to some degree of stability. Succeeding passes of the roller should then gradually progress toward the edges of the spread. The uncompacted edge provides initial confinement during the first pass, thus minimizing lateral movement of the mix. After the central portion of the spread has been compacted, the mix will support the roller and allow the edge to be compacted without lateral movement.

With static, steel-wheeled rollers the operation should always progress with the drive wheel forward in the direction of paving. This is especially important in breakdown rolling since the greatest percentage of compaction occurs during that pass. A main reson why breakdown rolling should be done with the drive wheel is that there is a more direct vertical load applied by this wheel than the tiller wheel (Figure 7.11).

If the breakdown pass of the roller is made with the tiller wheel forward, the pushing force and the weight is slightly ahead of vertical, causing material to push up in front of the wheel. The greater weight of the drive wheel carries out the compaction while the turning force tends to tuck material under the front of the wheel.

There are exceptions to rolling with the drive wheel forward, however. They usually occur when superelevations are being constructed or if the grade on which the asphalt mix is being placed is excessive. The exceptions occur when, due to these high grades, the drive wheel of the roller begins to chatter on the mat, causing displacement of the mix and a very rough surface. In these cases the roller must be turned around to allow the tiller wheel to partially compact the material so that the drive wheel can then proceed over it.

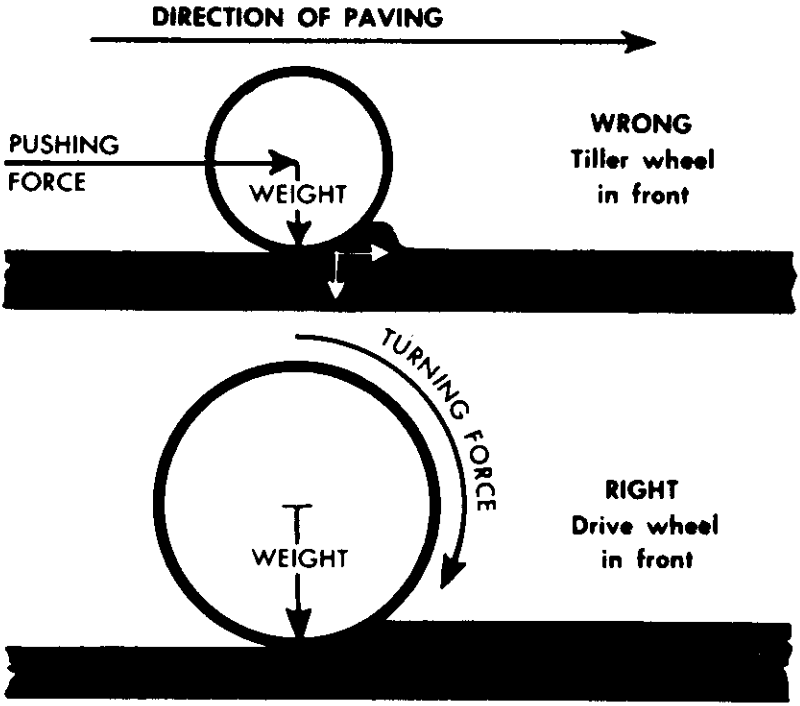


Figure 7.11 Forces Acting When Tiller Wheel or Drive Wheel Is Forward.

Intermediate Rolling

Secondary or intermediate rolling should closely follow breakdown rolling while the asphalt mix is still plastic and at a temperature that will result in maximum density.

Pneumatic-tired or vibratory tandem rollers may be used for intermediate rolling. When using pneumatic rollers, keeping the tires hot is the most effective means of preventing pickup. Applying a small amount of non-foaming detergent or water soluble oil on the wetting mat of a pneumatic-tired roller at the beginning of rolling operations helps prevent asphalt from sticking to the tires until they warm up.

Pneumatic-tired rollers have several advantages:

1. They provide a more uniform degree of compaction than steel-wheeled rollers;

2. They improve the seal near the surface, thus decreasing the permeability of the layer; and

3. They orient the aggregate particles for greatest stability, as high pressure truck tires do after using the asphalt surface for some time.

Tire contact pressures should be as high as possible without causing displacement of mix that cannot be remedied in the final rolling.

Pneumatic-tired rolling should be continuous after breakdown rolling until all of the mix placed has been thoroughly compacted. At least three coverages should be made. Turning of pneumatic-tired rollers on the paving mix should not be permitted unless it does not cause undue displacement.

Vibratory tandem rollers—of proper static weight, vibration frequency and amplitude—are used to provide required densities with fewer coverages than static-weight tandem or pneumatic-tired rollers (or combinations of the two).

Regardless of the type of rollers used, the rolling pattern should be developed in the same manner as for the initial, or breakdown rolling. This pattern should be continued until the desired compaction is obtained.

Finish Rolling

Finish rolling is done primarily for the aesthetic improvement of the surface. It should be accomplished with steel-wheeled tandems, static-weight or vibratory tandems (without using vibration), while the material is still warm enough for removal of roller marks.

7.4 Checking the Finished Pavement

Surface Texture

Defects in surface texture can be due to errors in mixing, handling, paving or compacting. An asphalt mixture that is defective due to improper mixing, handling or placing should be removed and replaced prior to compaction. Defects that show up during compaction and that cannot be corrected by additional rolling should be replaced with fresh hot asphalt mixture and compacted before the surrounding mat cools below 85° C (185° F). Care must be taken to ensure that proper grade and surface tolerance are maintained within any repaired area.

Surface Tolerance

The importance of staying within the surface tolerance for smoothness cannot be overemphasized. For modern high-speed traffic, most specifications allow a transverse variation of no more than 6 mm in 3 m (¼ in. in 10 ft), and a longitudinal variation of no more than 3 mm in 3 m (⅛ in. in 10 ft). Whatever surface tolerance is specified and required, efforts are made to obtain the same tolerance in each course being placed.

After each spread and before the next layer is placed, the surface must be checked with a straightedge to determine if the specified smoothness is being obtained. Two types of straightedges are in general use. The most basic is a wood or metal straightedge that is laid on the pavement so that irregularities can be measured from it. The other type is a rolling straightedge (Figure 7.12), which measures irregularities on a gauge as the inspector pushes it along. Any irregularities that vary more than the tolerance set by the specifications are corrected before the next layer is placed.

Irregularities in the lower courses can usually be corrected by removing or adding material. In the surface course, the entire affected area should be removed promptly and sufficient new material placed to form a true and even surface. If the required smoothness is obtained in the first course of plant mix laid, the subsequent courses can usually be placed uniformly by simply setting the paver for the thickness desired.

Pavement Density

Pavement density is determined to be acceptable or unacceptable by comparing it to the target density established in the laboratory from the field sample.

The three basic methods for determining target density are: percent of laboratory density, percent of theoretical maximum density, and test section

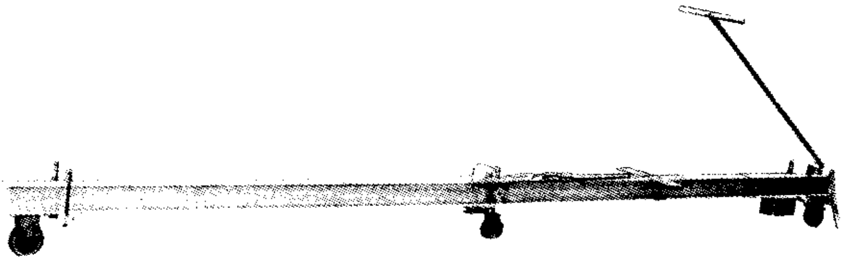


Figure 7.12 Rolling Straightedge for Checking Surface Tolerance.

(control strip) density. The intent of all three methods is to obtain a compacted pavement which will contain a specified void content.

- *Percent of Laboratory Density*

  This method is particularly applicable to Marshall compaction procedures on large projects where field laboratories are employed. A target density is determined for each lot or unit of mix production (usually one day's production) by taking the average density of four or more laboratory-prepared specimens taken randomly from trucks delivering mix to the job site. The specimens are compacted in the Marshall apparatus in accordance with ASTM D 1559 or AASHTO Method T 245 with two exceptions.

  1. The temperature of the mix should approximate the temperature at the paver with no reheating being permitted; and
  2. The number of compactive blows (35, 50 or 75) should be the same as was used in the mix design.

  The advantage of this procedure is that the resulting target densities will closely represent actual daily mix production and compensate for slight variations in the mix that occur from day to day.

- *Percent of Theoretical Maximum Density*

  In this method the target density is set as a percentage of what the unit weight of the mix would be if it were densified to a totally voidless mass. This theoretical maximum density is determined by ASTM D 2041 or AASHTO Test Method T 209 commonly known as the "Rice Test" or Maximum Effective Specific Gravity.

- *Test Section (control strip) Density*

  The target density is determined by constructing a control strip of pavement at the beginning of each lift or course being placed. The control strip is part of the paving project itself. It should be at least 150 m (500 feet) long and constructed to the same width and thickness as the rest of the course it represents.

The contractor places and compacts the control strip with the equipment and the rolling pattern and at the temperature that he proposes to use for the job.

Compaction begins as soon as possible after the mix is placed and continues until no appreciable increase in density is obtained and/or until the mix cools to 85° C (185° F). The target density is determined by averaging the results of a specified number of density tests taken at randomly selected sites within the control strip.

If the target density of the control strip is typically less than 92 percent of maximum theoretical density or 96 percent of laboratory prepared specimens of the same mix, densification is considered inadequate. (This target density is recommended by the Asphalt Institute; agency specification may differ.) A new control strip should be constructed incorporating necessary changes in compaction temperature, equipment and/or rolling procedures.

Additional Procedures

Sampling and Testing

Testing to verify field compaction can be done by either obtaining cores or sawed samples from the pavement or testing with a nuclear density gauge (Figure 7.13). Nuclear density readings should be correlated with core densities. A new correlation factor is needed for each mix design or significant change in the mix.

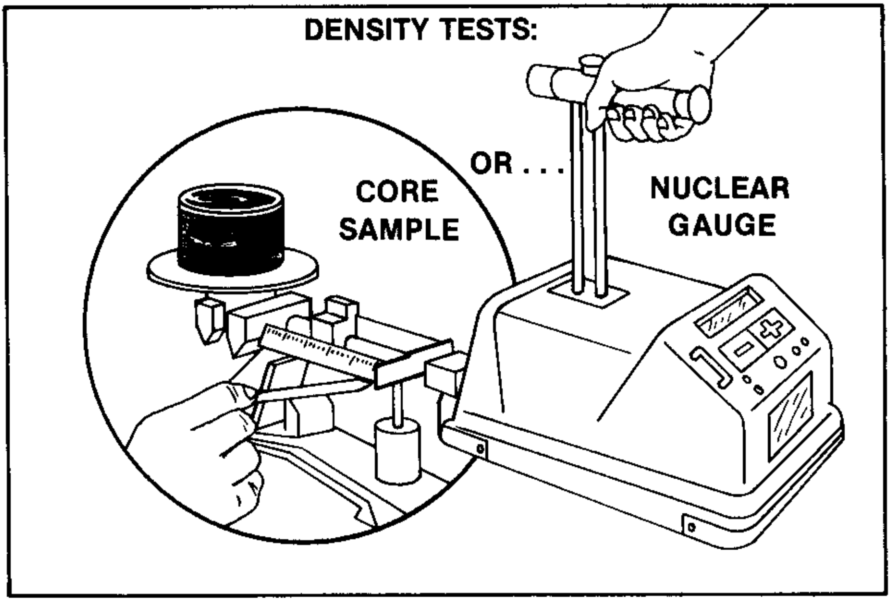


Figure 7.13 Density Tests.

Cores are cylindrical samples that are cut out of the pavement. The cores are tested in a laboratory to verify the density of the mat.

Nuclear density gauges have advantages over core sampling. They are fast, easy to use and are non-destructive.

Testing by either method is done on a random sampling basis, with a minimum of a specified number of density tests or pavement cores being tested for each lot of mix (usually one day's production). The *average* of the densities obtained by the tests must meet one or more of the following typical criteria depending on the method used to establish target density: 96 percent of laboratory density; 92 percent of maximum theoretical density; 99 percent of control strip density.

Samples obtained to verify density should be removed with a diamond tipped power saw or coring machine (Figure 7.14) to minimize damage or distortion of the pavement and samples. If the pavement has not cooled to ambient temperature for the full depth of the sample, ice or dry ice or even CO_2 have been used to expedite the cooling process.

Care must be exercised in obtaining and transporting samples from the field. Loose mix samples obtained for target density determination should be transported in insulated containers and as quickly as possible to minimize loss of heat prior to preparation of the specimen.

In addition to obtaining compacted samples for density measurements, it may occasionally be necessary to sample the mixed material on the road, for extraction and gradation tests, before it has been compacted. These samples may be taken from the hopper of the paver or from the bed of the truck at a minimum of four different points and blended into a composite sample. It should always be remembered that a test is no better than the sample, and that it is imperative that the sample be representative of the lot being tested.

Records and Reports

The inspector and engineer responsible for the paving should keep a daily diary, and from it prepare and submit a report at the end of each day's and each week's operation. These reports should be designed so that all supervisory personnel can keep abreast of an entire day's operations. At the completion of the work, the report may become a record of the construction progress. Changes in materials, methods, or conditions are noted in the diary and tied into station location on the roadway. Any time a change is made in gradation or proportions in the mix, it should be recorded with the location of the mix in the pavement. The daily diary is for the inspector's convenience during construction, but on completion of construction it becomes part of the permanent record of the project. Sample forms that may be used by inspectors are shown in Figures 7.15, 7.16 and 7.17. Metric units may be substituted for customary units.

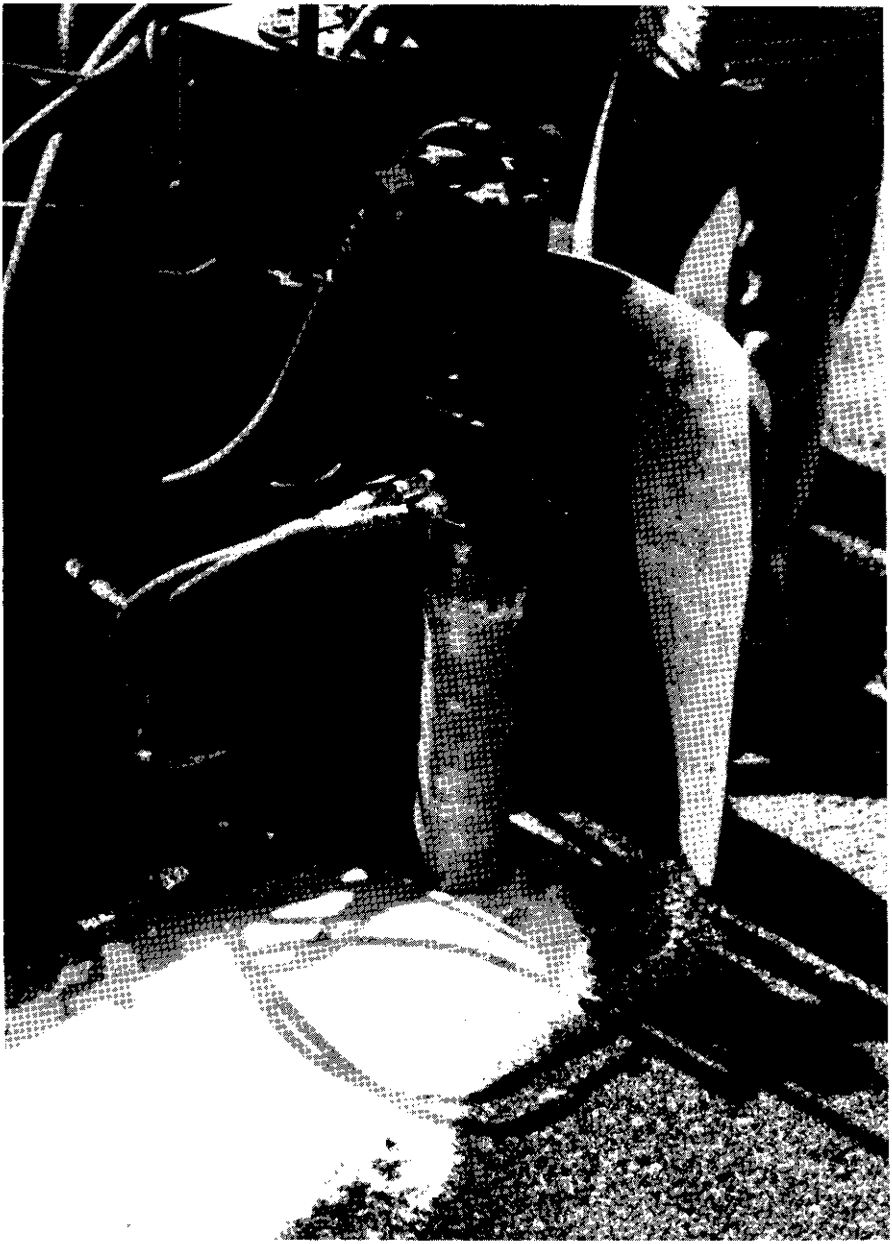
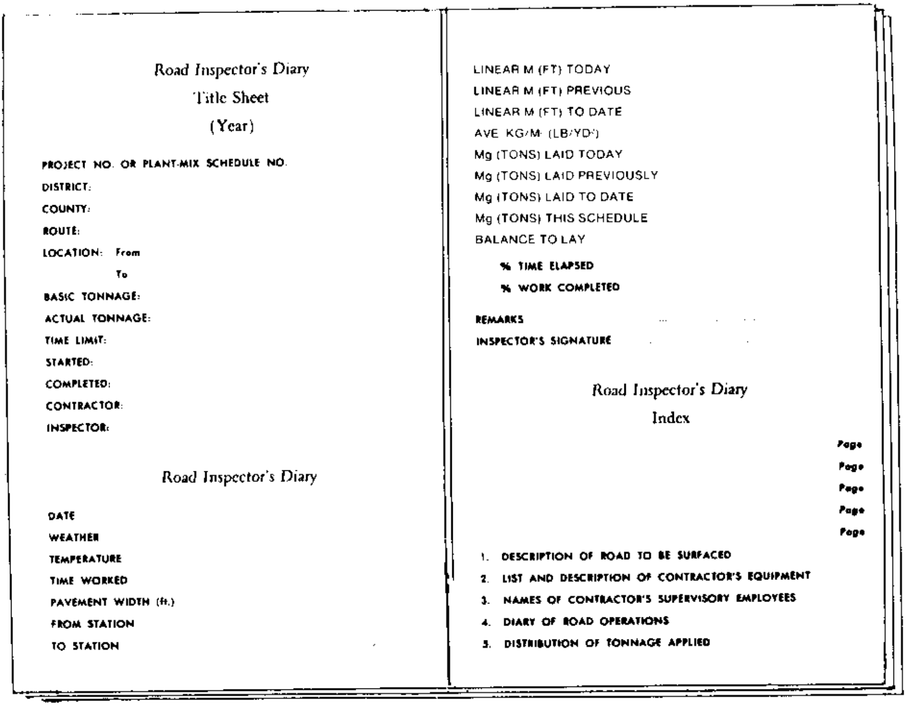


Figure 7.14 Taking Core Sample of Finished Pavement.



Road Inspector's Diary

Title Sheet

(Year)

PROJECT NO. OR PLANT-MIX SCHEDULE NO.
DISTRICT:
COUNTY:
ROUTE:
LOCATION: From
To
BASIC TONNAGE:
ACTUAL TONNAGE:
TIME LIMIT:
STARTED:
COMPLETED:
CONTRACTOR:
INSPECTOR:

Road Inspector's Diary

DATE
WEATHER
TEMPERATURE
TIME WORKED
PAVEMENT WIDTH (ft.)
FROM STATION
TO STATION

LINEAR M (FT) TODAY
LINEAR M (FT) PREVIOUS
LINEAR M (FT) TO DATE
AVE. KG/M^2 (LB/YD^2)
Mg (TONS) LAID TODAY
Mg (TONS) LAID PREVIOUSLY
Mg (TONS) LAID TO DATE
Mg (TONS) THIS SCHEDULE
BALANCE TO LAY
% TIME ELAPSED
% WORK COMPLETED

REMARKS
INSPECTOR'S SIGNATURE

Road Inspector's Diary

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1. DESCRIPTION OF ROAD TO BE SURFACED
2. LIST AND DESCRIPTION OF CONTRACTOR'S EQUIPMENT
3. NAMES OF CONTRACTOR'S SUPERVISORY EMPLOYEES
4. DIARY OF ROAD OPERATIONS
5. DISTRIBUTION OF TONNAGE APPLIED

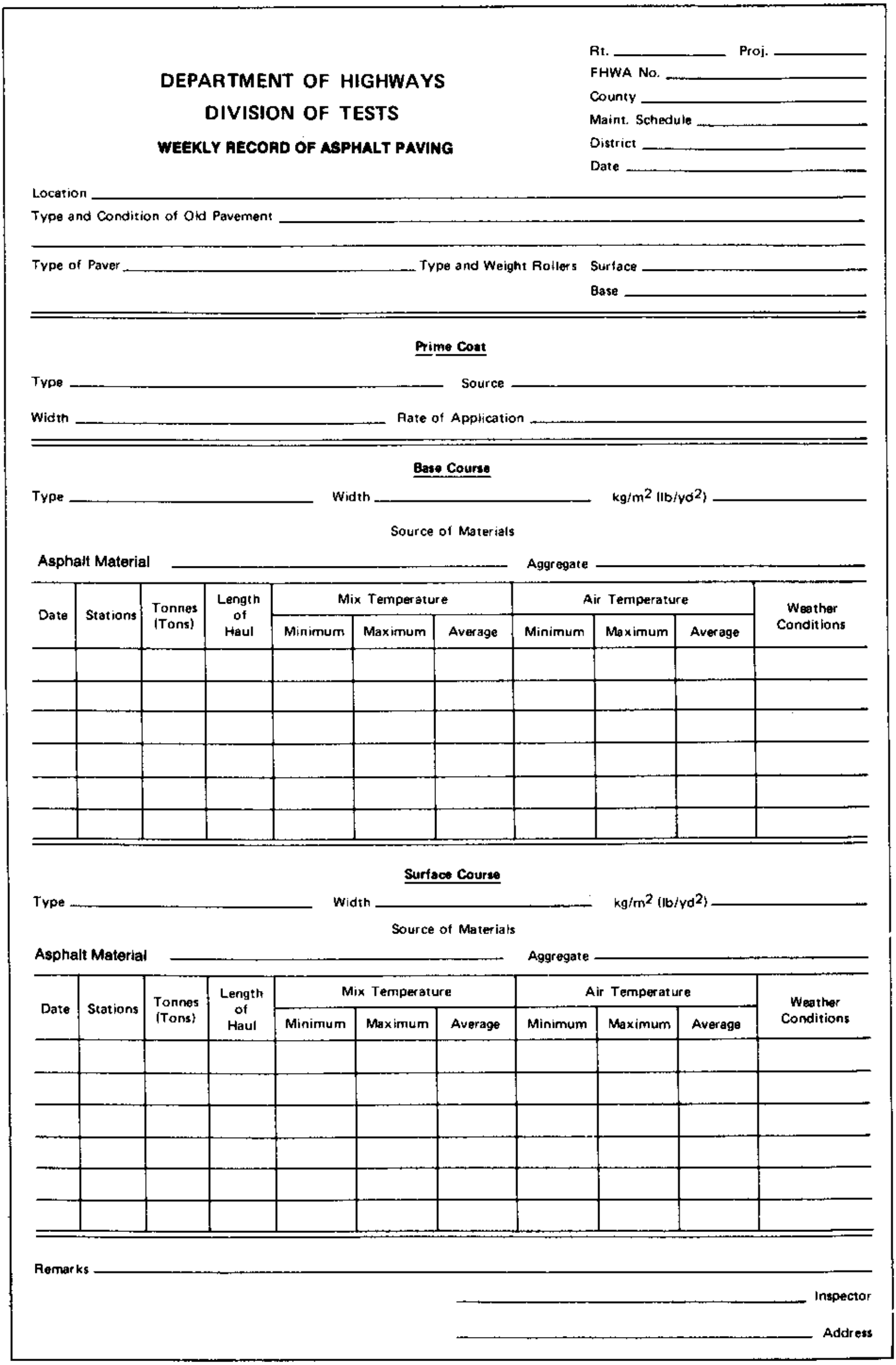
Figure 7.15 Pavement Inspection Diary.

Traffic Control

Traffic should be kept off the surface of newly laid mix until it has cooled enough not to become marked. Where traffic is maintained in partial-width construction, it is necessary to have sufficient, properly placed, warning signs so that traffic will keep off the freshly laid mixture. The work should be planned so that traffic is not greatly inconvenienced by having to travel on long one-way lanes. If one-way traffic is necessary over sections having insufficient sight distance or lack of shoulder turnouts, it should be controlled by a shuttle flag or pilot truck.

Good Housekeeping

Good housekeeping should be maintained during construction. Discarded mix, boards, trash, discarded parts of equipment, discarded hand tools, etc., should not be left on the right-of-way. After paving and compaction operations have been completed and before personnel leave the job site, a final cleanup should be made.



Rt. ______ Proj. ______
FHWA No. ______
County ______
Maint. Schedule ______
District ______
Date ______

DEPARTMENT OF HIGHWAYS
DIVISION OF TESTS
WEEKLY RECORD OF ASPHALT PAVING

Location ______
Type and Condition of Old Pavement ______
Type of Paver ______ Type and Weight Rollers Surface ______
Base ______

Prime Coat

Type ______ Source ______
Width ______ Rate of Application ______

Base Course

Type ______ Width ______ kg/m^2 (lb/yd^2) ______

Source of Materials

Asphalt Material ______ Aggregate ______

Date	Stations	Tonnes (Tons)	Length of Haul	Mix Temperature			Air Temperature			Weather Conditions
				Minimum	Maximum	Average	Minimum	Maximum	Average	

Surface Course

Type ______ Width ______ kg/m^2 (lb/yd^2) ______

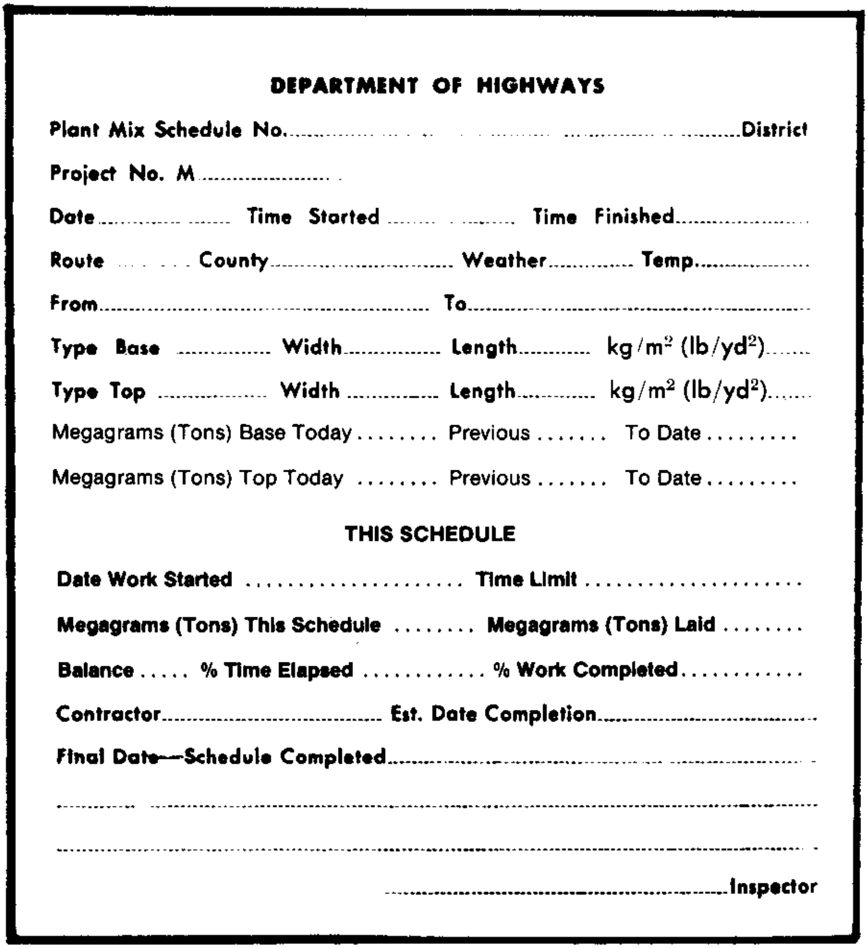
Source of Materials

Asphalt Material ______ Aggregate ______

Date	Stations	Tonnes (Tons)	Length of Haul	Mix Temperature			Air Temperature			Weather Conditions
				Minimum	Maximum	Average	Minimum	Maximum	Average	

Remarks ______
______ Inspector
______ Address

Figure 7.16 Road Inspector's Weekly Report.



DEPARTMENT OF HIGHWAYS

Plant Mix Schedule No. .... District

Project No. M ....

Date .... Time Started .... Time Finished ....

Route .... County .... Weather .... Temp. ....

From .... To ....

Type Base .... Width .... Length .... kg/m^2 (lb/yd^2) ......

Type Top .... Width .... Length .... kg/m^2 (lb/yd^2) ......

Megagrams (Tons) Base Today .... Previous .... To Date ....

Megagrams (Tons) Top Today .... Previous .... To Date ....

THIS SCHEDULE

Date Work Started .... Time Limit ....

Megagrams (Tons) This Schedule .... Megagrams (Tons) Laid ....

Balance ..... % Time Elapsed .... % Work Completed ....

Contractor .... Est. Date Completion ....

Final Date—Schedule Completed ....

....

....

.... Inspector

Figure 7.17 Road Inspector's Daily Report.

Chapter 8
Asphalt Surface Treatments

Asphalt surface treatments are versatile, widely used forms of asphalt construction and maintenance. As the following sections indicate, there are several forms of treatments and seal coats. Properly-designed, they are economical, easy to place and long lasting. They seal and add life to road surfaces, but each type has one or more special purposes. A surface treatment is not a pavement in itself. Rather, it provides a protective cover that helps to resist traffic abrasion and provides a waterproof cover over the underlying structure. It adds little or no load-carrying strength and therefore is not normally taken into account in computing the load limit of a pavement. While a surface treatment can provide an excellent surface if used for the correct purpose, it is not a solution for all pavement problems. A clear understanding of the advantages and limitations of asphalt surface treatments is essential for best results. It is vital that a careful study of traffic requirements, along with an evaluation of the condition of existing materials and pavement layers, be made prior to deciding if a surface treatment should be used.

SECTION 8.1 Asphalt Surface Treatment Types

☐ Types
☐ Functions of Surface Treatment

SECTION 8.2 Asphalt-Aggregate Surface Treatments

☐ Types
☐ Materials
☐ Asphalt Distributor
☐ Aggregate Spreaders
☐ Other Equipment
☐ Surface Treatment Design
☐ Construction Procedure
☐ Special Asphalt-Aggregate Surface Treatments

SECTION 8.3 Other Surface Treatments

☐ Introduction
☐ Sprayed Asphalt Treatments
☐ Emulsified Asphalt Slurry Seal

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2. *Asphalt Surface Treatments-Construction Techniques, ES-12,* Asphalt Institute.
3. *A Basic Asphalt Emulsion Manual, MS-19,* Asphalt Institute.

VISUAL AID

1. *Asphalt Emulsion Spray Applications*, VA-1V, VHS tape (color, sound, 23 min), Asphalt Institute.

8.1 Asphalt Surface Treatments

Types

Asphalt Surface Treatment Defined

Asphalt surface treatment, or simply surface treatment, is a broad term embracing several types of asphalt applications with or without a cover of mineral aggregate to any type road or pavement surface. It is usually less than 25 mm (1 in.) thick. An asphalt seal coat, or simply seal coat, is a thin asphalt surface treatment applied to a paved surface.

Types of Asphalt Surface Treatments

Surface treatments range from single, light applications of diluted emulsified asphalt to multiple surface courses made up of alternate applications of asphalt and aggregate. One type of asphalt-aggregate mix, a slurry seal, is also considered as a surface treatment. All surface treatments can add life to existing road surfaces. Each type has one or more specific purposes. The following is a classification of asphalt surface treatments by application or use:

1. Sprayed asphalt and aggregate surface treatment (Section 8.2):
    - Single surface treatment
    - Multiple surface treatment
    - Sand seal
2. Sprayed asphalt treatment (Section 8.3):
    - Prime coat
    - Dust laying
    - Road oiling
    - Tack coat
    - Fog seal
3. Emulsion slurry seal (Section 8.3):

Functions of Surface Treatment

A surface treatment by itself is not considered a pavement. Basically, it provides a waterproof cover over the existing pavement surface and provides resistance to abrasion by traffic. Specific functions of surface treatments are:

- To provide long-lasting, economical surfaces for granular base roads having light and medium traffic volumes
- To prevent surface water from penetrating granular bases or old pavements that have become weathered or cracked
- To plug voids, coat and bond loose aggregate particles in pavement surfaces

- To renew a surface and restore skid resistance to traffic-worn pavements in which the surface aggregate has become polished
- To restore weathered surfaces and give new life to oxidized pavement surfaces
- To provide temporary cover in cases of delayed pavement construction or planned stage construction
- To control dust on low volume roads
- To promote adherence of subsequent asphalt courses to granular bases (prime coats)
- To ensure a bond between subsequent pavement layers being placed or overlays of existing surfaces (tack coats)
- To improve the aesthetics of an existing pavement

8.2 Asphalt-Aggregate Surface Treatments

Types

Single Surface Treatment

A single surface treatment is used as a wearing and waterproofing course. It consists of a sprayed application of asphalt immediately covered by a single layer of aggregate of uniform size. The thickness of the treatment approximates the nominal maximum size aggregate particles used in the particular surface treatment.

Multiple Surface Treatment

A multiple surface treatment provides a wearing and waterproofing course of greater durability than a single surface treatment, and may add some strength to the pavement structure. It consists of two or more alternate applications of asphalt and aggregate. The nominal top size of the cover aggregate for each successive course should not be more than one-half that of the preceding one.

Sand Seal

A sand seal is an application of asphalt material covered with fine aggregate. It may be used to improve the skid resistance of slippery pavements or to seal against air and water infiltration. Construction procedures for sand seals are similar to those for single surface treatments. Typically, asphalt is sprayed onto the surface to be treated followed by an application of sand.

Materials

Asphalt

In selecting the proper type and grade of asphalt for a surface treatment, consideration is given to the following:

1. Temperature of the surface to which the treatment will be applied,
2. Air temperature,
3. Humidity and wind,
4. Condition of the surface, and
5. Type and condition of the aggregate to be applied.

The correct type and grade of asphalt for the surface treatment will:

1. When being applied, be fluid enough to spray and cover the surface uniformly, yet viscous enough to remain in a uniform layer and not puddle in depressions or run off the crown at grade;
2. After application, retain the required consistency to wet the applied aggregate;

3. Cure and develop adhesion quickly;
4. After rolling and curing, hold the aggregate tightly to the road surface to prevent dislodging by traffic; and
5. When applied in the proper amount, not bleed or strip with changing weather conditions.

Certain grades of asphalt cements and emulsified asphalts satisfy these requirements. In hot, dry weather, soft grades of asphalt cement may be used if dry aggregate is placed immediately after spraying. However, rapid-setting emulsifed asphalts are usually the most suitable for surface treatments.

The type and grades of asphalts most suitable for single and multiple surface treatments are:

- RS-1, RS-2; CRS-1, CRS-2; MS-1, HFMS-1.
- AC 2.5, AC-5, (AR-1000, AR-2000; 200/300 or 120/150 pen).

Aggregate

Most hard aggregates, such as gravel, crushed stone, and crushed slag, can be successfully used for cover. The aggregate selected, however, must meet certain requirements of size, shape, cleanliness, and surface properties.

Aggregate should be as uniform as economically practicable so that the surface treatment will have essentially only one layer of aggregate. Generally, the largest size should be no more than twice the smallest size, with a reasonable tolerance in dimensions to allow for economical production. Aggregates having a maximum size of less than 13 mm (½ in.) in diameter provide a quieter and smoother riding surface than larger ones.

The ideal aggregate shape is cubical. A large amount of flat or elongated particles is undesirable because they may be completely covered by asphalt applied in a quantity sufficient to hold the cubical particles.

It is important that the aggregate be clean. If the particles are coated with dust, silt, or clay, the coating forms a film that prevents asphalt-aggregate adhesion. Good adhesion between aggregate and asphalt and retention of this adhesion are essential to a surface treatment's success. Adhesion, however, is influenced by many variables. Asphalt cement does not adhere well to wet aggregate. Best adhesion is obtained when aggregates are hot and dry. Some moisture can be tolerated in warm dry weather that promotes rapid drying when asphalt emulsion is used.

A range of asphalt and aggregate applications are provided later in this section (Table 8.1) as a guide to estimating the proper quantity of materials for a given aggregate size. Adjustments should be made for actual field and material conditions.

The Asphalt Distributor

The asphalt distributor is the most important piece of equipment on a surface treatment project. It is made specifically to apply the asphalt product uniformly and in specified quantities.

The asphalt distributor (Figure 8.1) consists of a truck-mounted or trailer-mounted insulated tank ranging in capacity from 3,000 to 20,800 litres (800 to 5,500 gal.). Most distributors are equipped with a heating system that will maintain the asphalt at the proper spray temperature.

The distributor has a power driven pump capable of handling products ranging from light cold applications of emulsified asphalt to heavy asphalt cements heated to spraying viscosity. At the back end of the tank is a system of spray bars with nozzles through which asphalt is forced under pressure to the surface of the road. These spray bars cover widths from 1.8 to 9.1 m (6 to 30 ft) in one pass, depending on their width and the pump's capacity.

The Circulating System

The circulating system has an engine driven pump which:

1. Fills the distributor tank,
2. Circulates material through the bar and tank,
3. Sprays material through the bar or hand spray,
4. Draws material back to the tank from the bar or hand spray,
5. Pumps material from the tank to outside storage and,
6. Transfers material from one storage tank to another.

The spray bar must have a constant and uniform pressure along the entire length for equal output from all nozzles.

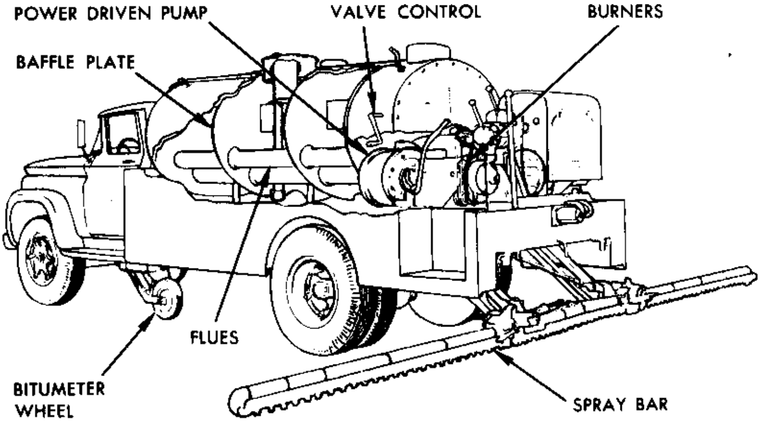


Figure 8.1 Asphalt Distributor.

Although the methods of maintaining pressure may vary, all distributors use gear-type pumps to deliver asphalt to the spray bar. On some distributors pressure is governed by a variable pump speed and on others by constant pump speed and a pressure relief valve.

The correct pump speed or pressure is that which neither atomizes the asphalt nor distorts the spray fan. A low pressure may result in streaking from a non-uniform discharge of material from the individual nozzles. A high pressure, besides atomizing the asphalt, may distort the spray fan. Manufacturers supply charts and data for proper pump speed or pressure for determining the discharge in volume per minute for each nozzle size.

The Asphalt Spray Bar

One of the most important parts of the distributor is the spray bar. For through the spray bar and nozzles, the proper quantity of asphalt is forced under pressure onto the road surface. To achieve good results, the correct size nozzles for the type and grade of asphalt and the application rate must be selected. Before use, nozzles should be checked for damage and proper setting.

The angle of the long axis of the nozzle openings (Figure 8.2) must be adjusted so that the spray fans do not interfere with each other. The nozzle angle varies according to the make of the distributor but is typically between 0.26 and 0.52 radians (15 and 30 degrees). It is important that all nozzles be set at the proper angle within close tolerances.

In addition to nozzle angle, the spray bar height above the pavement surface must be adjusted to ensure uniformity of the asphalt spread. It is important, too, that the correct height be maintained during the entire application. A spray bar at the incorrect height causes streaking. Also, if set too high, wind distortion of the spray fans may occur.

For example, the best results with 100 mm (4 in.) nozzle spacing come from an exact triple coverage of the spray fans (Figure 8.3). But with a 150 mm (6 in.) nozzle spacing, in order to get triple coverage, the height of the spray bar would be so great that it could be subject to wind distortion. In such cases, a double coverage pattern is used.

For best results, the height of the spray bar from the road surface should not vary more than 13 mm (½ in.) during application. Special steps must be taken to ensure this maximum permissible variance as the load lightens on the springs of the distributor truck. The frame of the distributor can be tied down to the axle during spreading runs. Many distributors have mechanical controls to maintain the proper height.

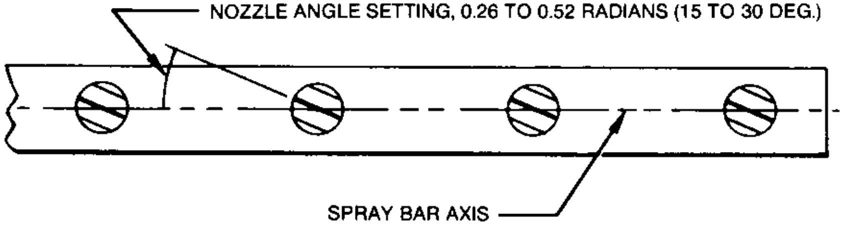


Figure 8.2 Proper Nozzle Angle Setting.

Controls

Controls include a valve system that governs the flow of material; a pump tachometer or a pressure gauge that registers pump output; and a bitumeter.

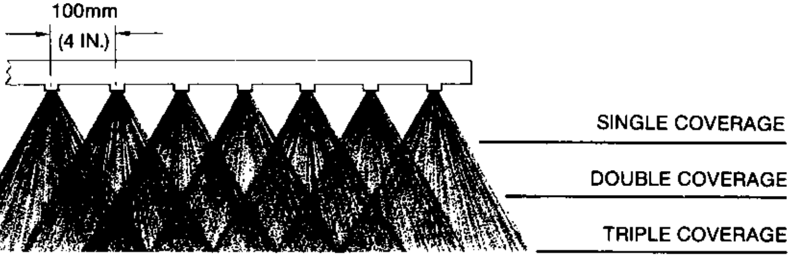


Figure 8.3 Overlapping Spray Fans.

The bitumeter is a rubber-tired wheel mounted on a retractable frame with a cable leading to a circular dial in the cab of the vehicle (Figure 8.4). Rate of travel in distance per minute, and the distance travelled are registered on the dial.

The bitumeter should be checked for accuracy at regular intervals. The bitumeter wheel must also be kept clean to ensure accurate registering of the truck speed. A buildup of asphalt on the wheel will produce an error.

To verify the correct rate of application, a shallow pan of known dimensions [often one square metre (yard)] can be placed in the distributors spray area. The asphalt collected in the pan should be weighed and converted to l/m^2 (gal/yd^2) for compaction with the specified rate.

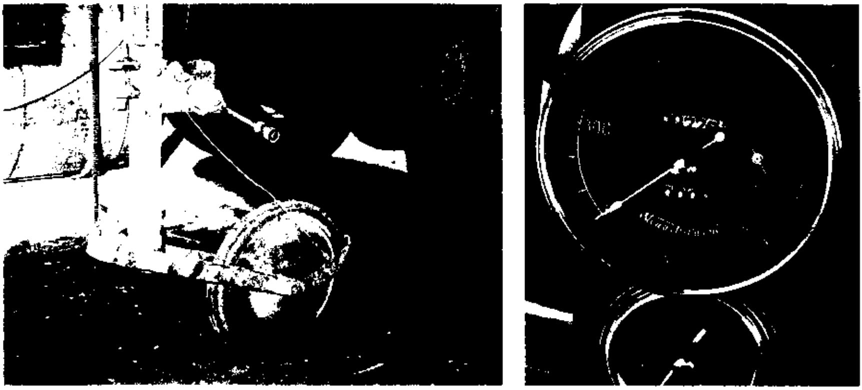


Figure 8.4 Bitumeter Wheel and Bitumeter Dial.

Aggregate Spreaders

The aggregate spreader is second only in importance to the asphalt distributor. Its function is to apply a uniform aggregate cover at a specified rate over the freshly sprayed asphalt membrane.

Aggregate spreaders for use in surface treatment construction consist of three basic types. One is the tailgate spreader which is connected directly to the tailgate of the truck. Another is the mechanical spreader which is also connected to the tailgate of the truck, but has wheels to support the aggregate spreader; and the third type is the mechanical, self-propelled spreader which actually pulls the truck and supplies the power for the spreading operation rather than being dependent on the truck itself.

There are several types of tailgate spreaders. The simplest is the vane spreader as shown in Figure 8.5. It consists of a steel plate with a series of vanes which spread the aggregate for the desired width on the road surface. There are tailgate spreaders that consist of a hopper with a feed roller which is activated by small wheels that contact truck wheels (see Figure 8.6).

Truck-mounted mechanical spreaders are hoppers on wheels which are hooked on to and are propelled by backing aggregate trucks as shown in Figure 8.7. Hoppers usually contain augers to distribute the aggregate the full width of the box. The wheels, which support the spreader and operate the auger, also turn a scored or roughened feed roll in the bottom of the hopper.

The feed roll ensures a uniform spread of material onto the road surface. There is an adjustable hitch on the front of the spreader by which it is attached to the aggregate truck. Mechanical spreaders also have controls to regulate the feed gates, the feed roll, the auger, and the truck hitch height.

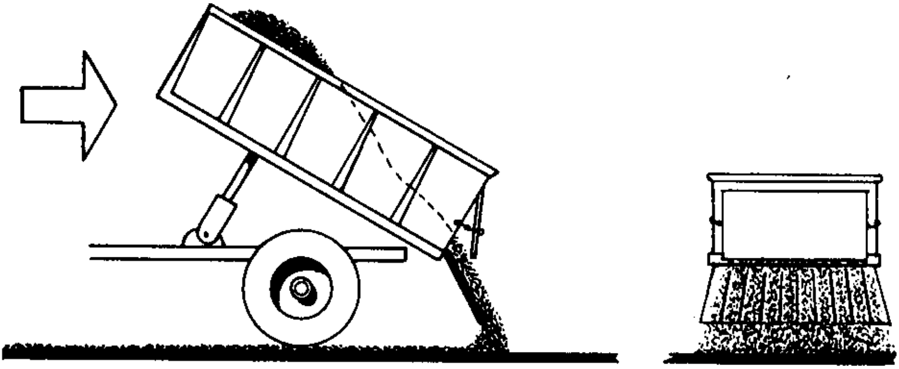


Figure 8.5 Tailgate Vane Spreader.

A self-propelled spreader is illustrated in Figure 8.8. This machine provides a very uniform and continuous distribution of the aggregate, and makes possible very high production rates. The self-propelled spreader is connected to the aggregate truck by means of a self-locking hook which, as the truck backs into the rear of the machine, locks on to the truck. The spreader then pulls the truck along the spread area. A receiving hopper is located at the rear of the machine. The material is moved from the receiving

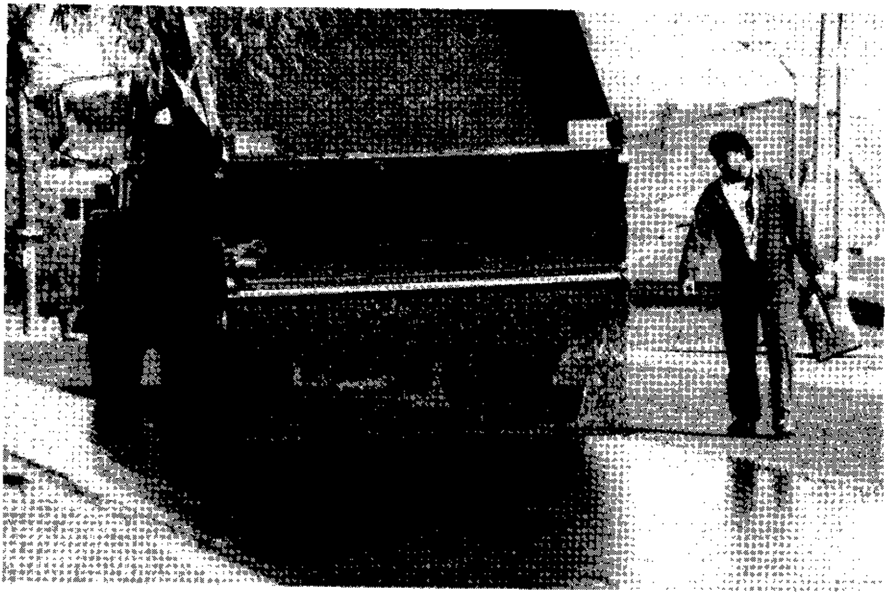


Figure 8.6 Hopper Type Tailgate Spreader.

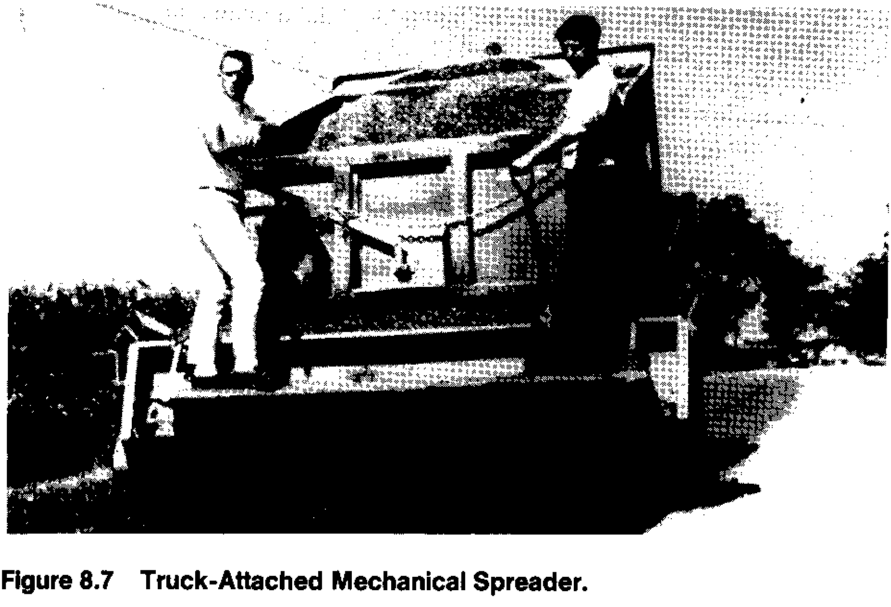


Figure 8.7 Truck-Attached Mechanical Spreader.

hopper to the spreading hopper by means of two individual, usually automatically-controlled, conveyor belts.

By adjusting the speed of each conveyor belt, the flow of aggregate into the hopper can be set to match the flow of the hopper. Material-level sensors in the spread hopper, combined with belt speed control assure the operator of a consistently full spread hopper without a second person on the machine.

With individually controlled gates, the operator can utilize as little as 150 mm (6 in.) of the width of the spreader to as much as the complete width of the spreader without leaving the operator's seat or having a second person on the machine. This enables the working of radii and intersections and variable roadbed widths. The material, when dropping into the spreading hopper, usually passes through a scalping screen at the top of the hopper. This enables the scalping off of any oversize material that may get into the receiving hopper from the truck. Material drops down into the spreading hopper and flows over a spread roll. The aggregate particles are then deposited onto the asphalt coated road surface.

The calibration and adjustment of all types of aggregate spreaders should be made according to manufacturers' instructions and operation manuals. However, here are some additional suggestions toward ensuring good results:

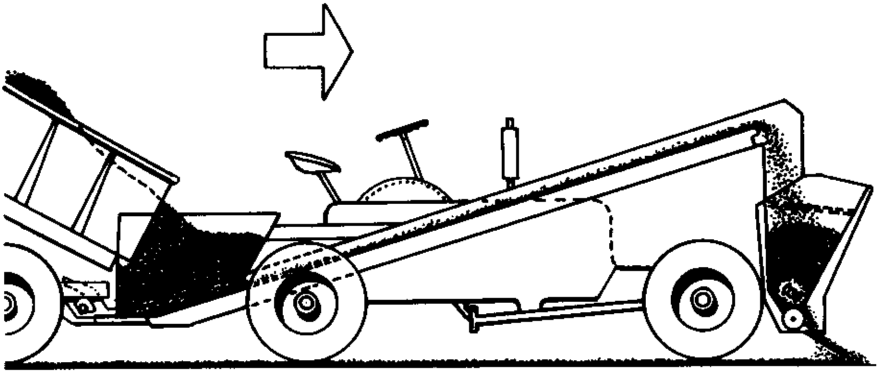
- A tachometer used as an aid in maintaining uniform spreader-speed has proven to be highly successful.
- Distribution rates can be closely controlled by marking off the length that each truck load of aggregate should cover.
- A quick check on the amount of aggregate being applied (rate of application) can be made by laying a 1 square metre (square yard) section of cloth or building paper on the pavement [or by supporting a shallow, 1 square metre (yard) box above the asphalt with nails or screws] and by passing over it with the spreader. The cloth, paper, or box is then carefully lifted, and the aggregate on it is weighed. This will give the application rate of the aggregate being spread.

Other Equipment

Cleaning Equipment

Unless the surface to be covered is completely clean, the asphalt may not adhere to the pavement. It is therefore necessary to clean the surface before spraying the asphalt emulsion. A power street sweeper, Figure 8.9 is recommended to pick up both dust and loose particles; but if one is not available a rotary power broom can be used. Flushing with water may be necessary when brooms are used, to meet clean air standards or to remove caked-on materials.

Power sweepers or brooms are also used to remove loose particles after the treatment is completed and the asphalt has properly cured. Only light broom pressure should be used in order to prevent dislodgment of the aggregate particles embedded in the asphalt membrance.



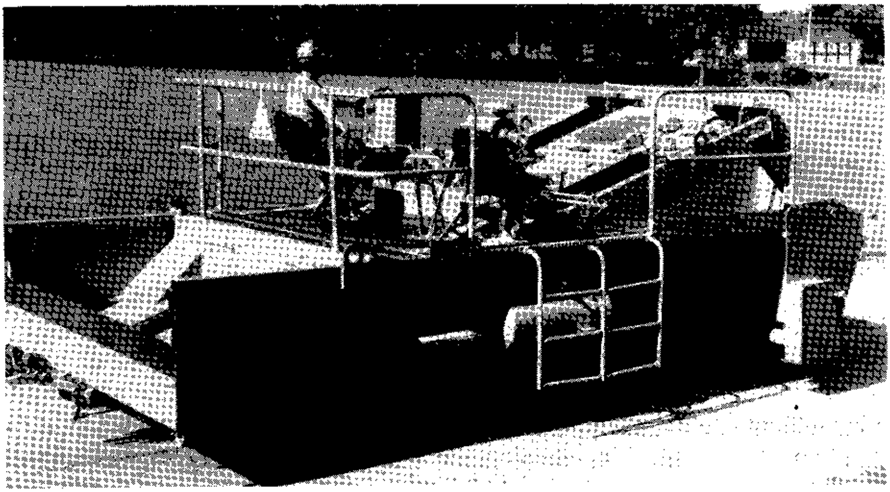


Figure 8.8 **(Top) Flow of Aggregate Through Self-Propelled Mechanical Spreader.**
(Bottom) Self-Propelled Mechanical Spreader.

Rollers

Seating of aggregate particles is an important part of the surface treatment operation. There are several types of compactors, but pneumatic-tired are usually used.

A self-propelled pneumatic-tired roller with tire pressures in the range of 415-620 kPa (60-90 psi) is recommended for use on asphalt-aggregate surface treatments. The tires force the aggregate firmly into the asphalt binder and to the flat side without crushing the particles. Steel-wheeled rollers bridge the smaller size particles and small depressions in the surface. Steel-wheeled rollers also may crush the softer particles so that degradation takes place even before traffic uses the new surface.

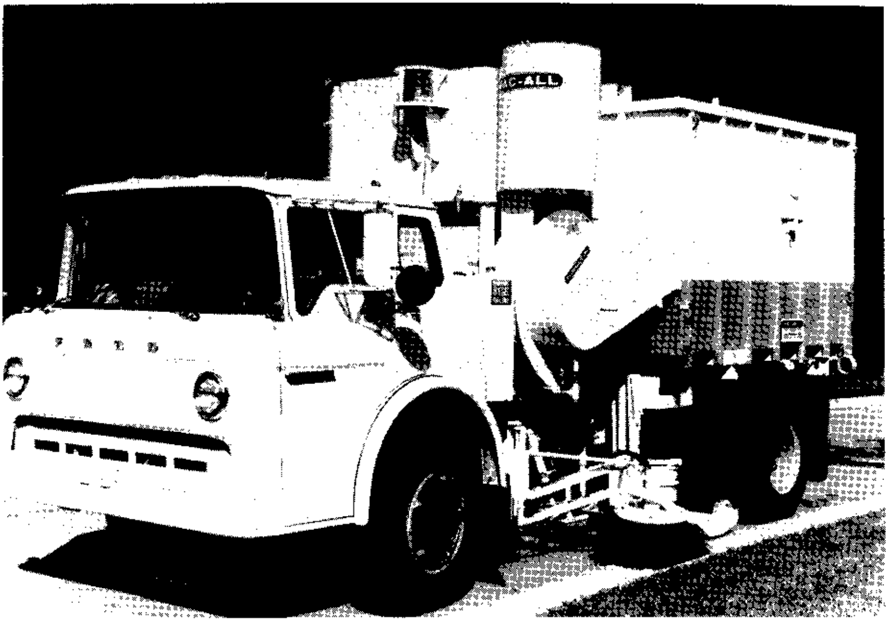


Figure 8.9 Power Sweeper. *(Courtesy Leach Co.)*

Surface Treatment Design

Single Surface Treatments

When a decision has been made that an asphalt-aggregate surface treatment is to be used, the next step is to find the proper rates of application for asphalt and aggregate. The objective is to produce a pavement surface one stone thick with enough asphalt to hold the aggregate in place, but not so much that it will bleed.

When a one-sized cover aggregate is dropped by a spreader on an asphalt film the particles will lie in an unarranged position. After compaction and considerable traffic, the particles will become oriented into their densest position with about 20 percent voids between the particles. It is desirable to fill these voids about two-thirds to three-fourths full with asphalt. A typical design will call for 70 percent of the voids filled. Because of the meniscus effect of the residual asphalt left on the aggregate upon the evaporation of the water when emulsified asphalt is used, the residual asphalt can be reduced to 55 to 60 percent of the voids between the aggregates under average conditions.

There are several theoretical procedures for determining the quantity of cover aggregate. These usually involve determining the average least dimension, the voids in the cover aggregate and the bulk specific gravity.

Mathematical calculations, coupled with laboratory testing, are usually employed in determining the required quantities of asphalt and aggregate.

Rather than presenting a complex means of making these determinations, Table 8.1 is presented. This table gives a range of asphalt and aggregate applications with respect to the specific size of aggregate being used.

The suggested quantities of asphalt in Table 8.1 are for the averge range of conditions that include primed granular bases and old pavement surfaces. The quantities and types of materials may be varied according to local conditions and experience or when actual laboratory tests are performed.

A rather simple way of estimating the quantity of aggregate is simply to spread the aggregate to be used over an area of 1 square metre (1 yd^2). A pan 1 × 1 metre × 25 mm (3 × 3 ft. × 1 in.) deep is suggested, as this will also allow determination of the asphalt quantity. Place aggregate in the densest condition anticipated to exist in the field after the surface treatment has been subjected to traffic. In order to do this it is necessary to have a good visual image of the finished product. Determine the weight of the aggregate required, and that weight will be the spread rate of the aggregate, in kilograms per square metre (pounds per square yard).

With the aggregate carefully arranged as described, fill the pan with water until the surface of the water comes just to the top of the aggregate. Measure this volume of water and use approximately two-thirds of that volume as the asphalt quantity required (adjusted, of course, by Note b of Table 8.1).

Multiple Surface Treatments

There are several arbitrary design methods for multiple surface treatments. In the method shown here, each course is designed as though it is a single surface treatment. For each succeeding course the nominal top size of cover aggregate should be not more than one-half the size of that for the previously placed course. No allowance is made for wastage. Also, after the first course, no correction is made for underlying surface texture.

Asphalt quantities for each course are added together and 40 percent of the total should be applied for the first application and 60 percent for the second application in a double surface treatment. In a triple surface treatment, 30 percent of the total should be applied for the first application, 40 percent for the second application and 30 percent for the third application.

In multiple surface treatments, the first course of cover aggregate generally determines the thickness. Subsequent courses partially fill the upper voids in the previously placed courses. (See Tables 8.2 and 8.3).

Construction Procedure

Sequence of Operations

The sequence of operations is basically the same for all types of surface treatment construction. The usual order is as follows:

1. Survey the condition of the surface to be treated;
2. Patch potholes and repair damaged areas in existing pavement;

Table 8.1 Quantities of Asphalt and Aggregate for Single Surface Treatments

	Nominal Size of Aggregate	AASHTO Size No.	Quantity of(a) Aggregate kg/m^2 (lbs/yd^2)	Quantity of(b) Asphalt l/m^2 (gal/yd^2)	Type and Grade of Asphalt
1	19.0 mm to 9.5 mm	6	22-27	1.6-2.0 (0.35-0.45)	Asphalt Cement(c)
	(¾ to ⅜ in.)		(40-50)	1.8-2.3 (0.40-0.50)	RS-2, CRS-2
2	12.5 mm to 4.75 mm	7	14-16	0.9-1.4 (0.20-0.30)	Asphalt Cement(c)
	(½ in. to No. 4)		(25-30)	1.4-2.0 (0.30-0.45)	RS-1, RS-2 CRS-1, CRS-2
3	9.5 mm to 2.36 mm	8	11-14	0.7-1.1 (0.15-0.25)	Asphalt Cement(c)
	(⅜ in. to No. 8)		(20-25)	0.9-1.6 (0.20-0.35)	RS-1, RS-2 CRS-1, CRS-2
4	4.75mm to 1.18 mm (No. 4 to No. 16)	9	8-11 (15-20)	0.7-0.9 (0.15-0.20)	RS-1, MS-1 CRS-1, HFMS-1
5	Sand	AASHTO M-6	5-8 (10-15)	0.5-0.7 (0.10-0.15)	RS-1, CRS-1 MS-1, HFMS-1

These quantities of asphalt cover the average range of conditions that include primed granular bases and old pavement surfaces. The quantities and types of materials may be varied according to local conditions and experience.

(a) The mass (weight) of aggregate shown in the table is based on aggregate with a specific gravity of 2.65. In case the specific gravity of the aggregate used is lower than 2.55 or higher than 2.75, the amount shown in the table above should be multiplied by the ratio that the bulk specific gravity of the aggregate used bears to 2.65.

(b) The lower application rates of asphalt shown in the above table should be used for aggregate having gradations on the fine side of the specified limits. The higher application rates should be used for aggregate having gradations on the coarser side of the specified limits.

It is important to adjust the asphalt content for the condition of the road, increasing it if the road is absorbent, badly cracked, or decreasing it if the road is "fat" with flushed asphalt.

Corrections for Surface Conditions

Texture	Litres/m^2	(Gal/Yd^2)
Black, flushed asphalt	-0.04 to -0.27	-0.01 to -0.6
Smooth, non-porous	0.00	0.00
Absorbent		
— slightly porous, oxidized	+0.14	+0.03
—slightly pocked, porous, oxidized	+0.27	+0.06
—badly pocked, porous, oxidized	+0.40	+0.09

(c) AC-2.5, AC-5, AR-1000, AR-2000; 200/300 pen., 120/150 pen.
(Note: In some areas persistent difficulty in retaining aggregate has been experienced with 200-300 penetration asphalt cements. Where this has occurred, the use of 200-300 penetration asphalt is not recommended.

Table 8.2 Quantities of Asphalt and Aggregate Per Square Metre (Square Yard) for Double Surface Treatment

	Nominal Size of Aggregate	Quantity of Aggregate kg/m² (lb/yd²)	Quantity of Asphalt l/m² (gal/yd²)
12.5mm (½") Thick			
1st Application*	9.5 to 2.36mm (⅜ in. to No. 8)	14-19 (25-35)	0.9-1.4 (0.20-0.30)
2nd Application	4.75 to 1.18mm (No. 4 to No. 16)	5-8 (10-15)	1.4-1.8 (0.30-0.40)
15.9mm (⅝") Thick			
1st Application*	12.5 to 4.75mm (½ in. to No. 4)	16-22 (30-40)	1.4-1.8 (0.30-0.40)
2nd Application	4.75 to 1.18mm (No. 4 to No. 16)	8-11 (15-20)	1.8-2.3 (0.40-0.50)
19.0mm (¾") Thick			
1st Application*	19.0 to 9.5mm (¾ to ⅜ in.)	22-27 (40-50)	1.6-2.3 (0.35-0.50)
2nd Application	9.5 to 2.36mm (⅜ in. to No. 8)	11-14 (20-25)	2.3-2.7 (0.50-0.60)

*If applied on untreated granular (aggregate) base use cutback asphalt in lieu of emulsions.

Table 8.3 Quantities of Asphalt and Aggregate Per Square Metre (Square Yard) for Triple Surface Treatment

	Nominal Size of Aggregate	Quantity of Aggregate kg/m² (lb/yd²)	Quantity of Asphalt l/m² (gal/yd²)
12.5mm (½") Thick			
1st Application*	9.5 to 2.36mm (⅜ in. to No. 8)	14-19 (25-35)	0.9-1.4 (0.20-0.30)
2nd Application	4.75 to 1.18mm (No. 4 to No. 16)	5-8 (10-15)	1.1-1.6 (0.25-0.35)
3rd Application	4.75mm to 150 μm (No. 4 to No. 100)	5-8 (10-15)	0.9-1.4 (0.20-0.30)
15.9mm (⅝") Thick			
1st Application*	12.5 to 4.75mm (½ in. to No. 4)	16-22 (30-40)	0.9-1.4 (0.20-0.30)
2nd Application	9.5 to 2.36mm (⅜ in. to No. 8)	8-11 (15-20)	1.4-1.8 (0.30-0.40)
3rd Application	4.75 to 1.18mm (No. 4 to No. 16)	5-8 (10-15)	0.9-1.4 (0.20-0.30)
19.0mm (¾") Thick			
1st Application*	19.0 to 9.5mm (¾ to ⅜ in.)	19-25 (35-45)	1.1-1.6 (0.25-0.35)
2nd Application	9.5 to 2.36mm (⅜ in. to No. 8)	11-14 (20-30)	1.4-1.8 (0.30-0.40)
3rd Application	4.75 to 1.18mm (No. 4 to No. 16)	5-8 (10-15)	1.1-1.6 (0.25-0.35)

*If applied on untreated granular (aggregate) base use cutback asphalt in lieu of emulsions.

3. Clean surface to be covered with power broom or other approved means;
4. Spray asphalt at specified rate and proper temperature (Tables 8.1-8.4);
5. Spread cover aggregate at specified rate immediately behind the asphalt spray application (emulsion still brown in color) to achieve maximum possible chip wetting;
6. Roll aggregate cover to seat particles in asphalt membrane.

Figure 8.10 shows a proper surface treatment operation. If a double or triple surface treatment is required, steps 4 through 6 should be repeated once or twice.

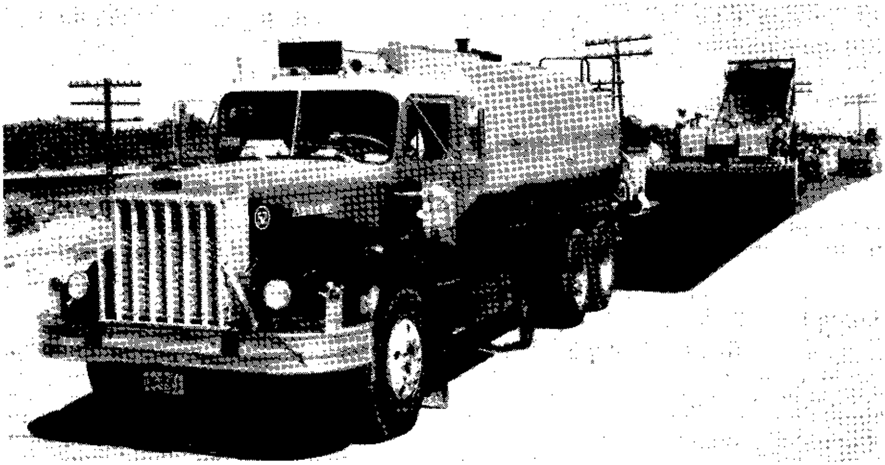


Figure 8.10 Surface Treatment Operation. *(Courtesy E. D. Etnyre Co.)*

Project Survey

Before any work is started, a thorough surface examination is made to determine needed repairs and to evaluate requirements for treatment. Soft spots that may result in future failures, as well as variations in width, cross section, and profile that would leave the road unsightly, are noted. Side drainage defects are especially critical. If an old surface is being reconditioned, look for potholes, cracked areas, depressions, slick or bleeding asphalt areas, absorbent areas, and other surface defects.

Roadway Surface Preparation

Some old pavements do not need repairs before thay are resurfaced. Others may require complete scarifying, mixing, and recompacting along with the removal of distressed material. However, most pavements need some patching and removal of excess asphalt.

Repairs and corrections must be made so the old surface will be in sound condition before treatment is begun. When surface repairs such as patching are performed, sufficient time should be allowed before surface treating to ensure consolidation under traffic.

Where the existing roadway is a granular surface and the cross section or profile is out of shape, the road should be scarified and new material added. To avoid thin layers that may flake off after the surface treatment has been placed, the new material should be mixed with the scarified material. It is then compacted in layers not more than 150 mm (6 in.) thick. Material in soft spots should be removed to the full depth and width necessary, and new base material added and compacted to a density equaling that of the surrounding base. A prime coat is then applied.

When all necessary repairs have been made, the surface must be cleaned immediately before the asphalt is sprayed. All hardened mud and other foreign matter must be removed and the surface thoroughly swept with a power broom.

Weather

Weather plays an important role in the success of surface treatments. The best time of year for a surface treating program is when weather conditions are most likely to be hot and dry during and (for some weeks) after treatment.

Many specifications require the air temperature be at least 10° C (50° F) before surface treatment operations begin. Some require the road surface temperature to be above 20° C (70° F) before work starts. No matter how hot the asphalt is when sprayed, it will cool to the temperature of the pavement surface in one minute or less.

Surface treatment should never be started when the surface is wet or when it is threatening to rain. The combination of water, fresh surface treatment, and traffic will result in loss of cover aggregate.

Equipment and Materials

Before work starts, all equipment should be examined to make sure it is in good working condition. The distributor operator must make sure that the spray bar is at the correct height above the surface and that all nozzles are set at the proper angle and are clear and spraying freely. The aggregate spreader should be checked to see that it is working properly.

Material delivery should be coordinated so that there are no delays after work starts. If the aggregate source is not near the project, a sufficient number of trucks are necessary to provide a continuous supply of material. Or, enough aggregate to finish the job should be stockpiled in advance near the job site.

Asphalt Spraying

Since the asphalt is sprayed on the roadway when constructing surface treatments, the temperature giving the most effective viscosity should be selected. The temperatures shown in Table 8.4 should be used as a guide. Each distributor has one or more thermometers for checking the asphalt temperature. The temperature used for spraying at a given spray-bar pressure should be that at which no fogging occurs when the asphalt material leaves the spray nozzle.

Table 8.4 Suggested Temperatures for Spraying Asphalt

Type and Grade of Asphalt	Spraying Temperature of Asphalt	
	Degree Celsius*	Degree Fahrenheit
Emulsified Asphalts		
CRS-1	50-85	125-185
CRS-2	50-85	125-185
RS-1	20-60	70-140
RS-2	50-85	125-185
MS-1	20-70	70-160
HFMS-1	20-70	70-160
Asphalt Cements	130+**	265+**

Notes:
*1. Exact conversions for °F are rounded to the nearest 5°C.
**2. The maximum temperature shall be below that at which fogging occurs.

Speed of the distributor and length of asphalt shot are determined before spraying starts. Distributor speed may be determined by:

Metric **Customary**

$$S = \frac{Q}{WR} \qquad S = \frac{9Q}{WR}$$

where

S = distributor speed, m/min (fpm);
Q = spray bar output, litre/min (gal/min);
W = spray width, m (ft);
R = rate of application, litre/m^2 (gal/yd^2).

Asphalt should be at the proper temperature for spraying viscosity. If it is, application at the correct rate presents no problem. The driver merely maintains the predetermined speed as indicated on the dial of the bitumeter.

The length of asphalt shot is determined before beginning each asphalt application. It is based on the number of loaded aggregate trucks on hand when operation begin. No more asphalt is applied then can be covered with aggregate within one minute. The length of spread may be determined by:

Metric **Customary**

$$L = \frac{T}{WR} \qquad L = \frac{9T}{WR}$$

where

L = length of spread, m (ft);
T = total litres (gallons) to be applied from the distributor to the surface;
W = spray width, m (ft); and
R = rate of application, litres/m^2 (gal/yd^2)

Once the length of the shot has been determined the distributor driver merely has to maintain a preset speed. Although the distributor contains several metering devices to indicate asphalt application rate, it is necessary that specific calculations be made following each shot. This can be done quite simply using the formula below. To check the rate of application, it is necessary that the number of litres (gallons) of asphalt sprayed be known along with the length and width of the spread.

Metric	Customary
$R = \frac{TM}{WL}$	$R = \frac{9TM}{WL}$

where

R = rate of application, litres/m² (gal/yd²);
T = total litres (gallons) spread from the distributor at spraying temperature (gauge stick reading before spread minus gauge stick reading after spread);
M = multiplier for correcting asphalt volume to basis 15.6° C (60° F), from temperature-volume tables;
W = width of spread, m (ft);
L = length of spread, m (ft).

Transverse Joints

Rough and unsightly transverse joints can be avoided by starting and stopping the asphalt and aggregate spread on building paper. The paper is placed across the lane to be treated so that the forward edge is at the desired joint location. The distributor—traveling at the correct speed for the desired application rate—starts spraying on the paper in order that the spray bar makes a full, uniform application when reaching the exposed surface. A second length of building paper is placed across the lane at the predetermined cutoff point for the distributor. This procedure gives a straight, sharp transverse joint. After the aggregate spreader has passed over it, the paper should be removed and disposed of properly.

For the next application, the leading edge of the paper is placed within 13 mm (½ in.) of the cut off line of the previously laid treatment. This prevents a gap between the two spreads.

Longitudinal Joints

Full-width applications of asphalt and aggregate will eliminate longitudinal joints. Typically, in most surface treatment work, however, the joint is unavoidable because traffic lanes must be maintained.

To prevent aggregate from building up on the longitudinal joint, the edge of the aggregate spread should coincide with the edge of the full thickness of applied asphalt. The full thickness of applied asphalt is narrower than the total width of applied asphalt by one-half the width of coverage of the nozzles on the end of the spray bar. This leaves a width where asphalt is present in partial thickness that will be overlapped when asphalt is applied in the adjacent lane. Then, when the aggregate is spread for the full width in the next lane, there is no buildup at the joint. The width of the asphalt strip left exposed varies [typically 150-200 mm (6-8 in.)], depending on the nozzle spacing and whether it is a triple- or double-lap spray pattern.

If possible, the longitudinal joint should be along the centerline of the pavement being treated. An established line helps to ensure a straight longitudinal joint.

Spreading Aggregate

All aggregate needed for the planned spread should be on hand before starting. When the distributor moves forward to spray asphalt, the aggregate spreader should follow immediately behind it. The asphalt must be covered within one minute. Otherwise, the increase in viscosity that takes place within that time may prevent good wetting and binding of the aggregate. It is also important that aggregate be spread uniformly at the specified rate. In a single application, aggregate normally does not stick more than one particle thick to the asphalt, so it is useless and wasteful to apply it at a rate greater than a single layer. Also, excess aggregate under the action of traffic can loosen embedded aggregate.

Excellent control is possible with mechanical spreaders. A uniform application rate can be assured with a properly adjusted spreader if a tachometer is used to maintain uniform speed. Another aid in controlling distribution rates is to mark the length that each truckload of aggregate should cover.

Excess aggregate, if placed in some areas, should be removed immediately with square-end shovels. In areas where application is insufficient, additional aggregate should be added immediately. With properly adjusted and operated aggregate spreaders, however, hand-work should not be necessary.

Rolling

Rolling seats the aggregate in the asphalt membrane and thus promotes the bond necessary to resist traffic stresses.

Pneumatic-tired rollers should be used on all surface treatment jobs. Pneumatic-tired rollers give uniform pressure over the entire area. Better results will be obtained if two self-propelled rollers are used with each aggregate spreader.

Pneumatic-tired rolling begins immediately after the cover material has been distributed and continues until the aggregate is properly seated in the asphalt membrane. As soon as the asphalt has a definite set or hardening, rolling is discontinued to prevent the bond between the surface and aggregate from being broken by the roller. Rolling begins at the outer edge of the treatment and proceeds in a longitudinal direction, working toward the center of the road. Each trip should overlap the previous trip by about one-half the width of the front wheels.

Excess Aggregate

Despite precautions, there is usually loose aggregate on the road surface after rolling is completed. Before the adjacent lane is covered with asphalt, loose aggregate is swept from along the joint and, if necessary, from the rest of the uncovered lane.

Aggregate that does not stick is often picked up by tires of fast-moving vehicles and thrown against following vehicles, often damaging headlights, windshields, and finish. Loose aggregate may be removed by lightly brooming with a rotary power broom. This brooming should be delayed until the asphalt has set and a good bond has developed. A good time to do brooming is during the cool of early morning.

Traffic Control

Traffic control is important to high-quality work and it must be maintained throughout the job. High-speed traffic over a fresh surface treatment displaces aggregate and produces a slick, black surface. Traffic should be detoured or allowed only in the lane not under construction. When work is completed and initial asphalt set has occurred, traffic is still maintained at less than 32 km/hr (20 mph) until the asphalt finally sets. The time until final setting varies with the weather. Traffic must be controlled in a manner that provides maximum safety for the workmen and least possible interruption of the work. Warning signs, flagmen, and a pilot truck leading vehicles past the work are safe and efficient means of controlling traffic.

Traffic control also extends to the hauling equipment. Aggregate hauling trucks are routed to the aggregate spreader in a direction opposite to the progress of the surface treatment operation. This prevents trucks from being turned on the freshly placed treatment. All trucks are required to turn around at a designated spot away from the new work.

Construction Procedures for Multiple Surface Treatments

Construction procedures for multiple surface treatments are essentially the same as those for single surface treatments, except that the process is repeated two or three times. The procedure for a double surface treatment consists of the following steps:

1. Surface preparation,
2. Placing first application of asphalt,
3. Placing first course of aggregate,
4. Rolling first course of aggregate,
5. Placing second application of asphalt,
6. Placing second course of aggregate,
7. Rolling second course of aggregate, and
8. Brooming off excess aggregate.

There should be curing between each treatment. The first treatment should be allowed to cure before placing the second application of asphalt (step 5). Under favorable conditions this period ranges from one to seven days. For a three-course treatment, steps 5, 6, and 7 are repeated for the additional application.

Special Asphalt-Aggregate Surface Treatments

Cape Seal

The term "Cape Seal" is attributed to the Cape Provincial Administration of South Africa. A Cape Seal can be defined as a single layer surface treatment (chip seal) followed by an emulsion mix slurry seal.

In order to have a successful Cape Seal project beyond following standard surface treatment and slurry seal specifications and methods, it is important that the surface treatment be placed as a single course only. The most critical element to avoid in a Cape Seal is an excess of slurry, as this can destroy the desired knobby surface texture. A cure time of four to ten days between placement of the surface treatment and subsequent slurry seal application should also be provided for, during which time regular brooming should occur to remove loose cover material or other foreign material that would prevent the adherence of the slurry. [See Table 8.5].

Table 8.5 Quantities of Asphalt and Aggregate Per Square Metre (Square Yard) for Cape Seal

12.5mm (½") Thick	L/m²-Asphalt (gal/yd²)	Kg/m²-Aggregate (lbs./yd²)	Kg/m²-Slurry Mixture (lbs./yd²)
Emulsion (RS-2, CRS-2)	1.4-2.0 (0.30-0.45)		
Cover Aggregate ASTM or AASHTO Size No. 7 12.5 to 4.75mm (½" to No. 4)		14-16 (25-30)	
Slurry Seal Type I			3-5.5 (6-10)

Sand Seal

Sand seal is defined as a spray application of asphalt emulsion followed with a light covering of fine aggregate, such as clean sand or screenings. Although this is a rather simple operation, it can be useful in correcting a number of pavement flaws. Usually, emulsion grades RS-1, CRS-1, MS-1 or HFMS-1 are used at a rate of about 0.70 to 0.90 litre/m² (0.15 to 0.20 gal/yd²). This is followed by about 5.5 to 8 kg/m² (10 to 15 lb/yd²) of sand, or screenings cover.

The sand seal is used primarily for the following purposes:

- To enrich a dry, weathered or oxidized pavement
- To prevent the intrusion of air and moisture
- To develop a skid-resistant surface texture
- To slow the rate of deterioration of an existing pavement.

8.3 Other Surface Treatments

Introduction

Descriptions

It was stated previously that all surface treatments seal and add life to road surfaces, but each type had one or more special purposes. Surface treatments remaining to be discussed are sprayed-asphalt type surface treatments and emulsion slurry seals.

Asphalt Selection

Table 8.6 suggests types of asphalt for use in various seal-type surface treatments. Included are surface treatments with cover aggregates (as already discussed).

Table 8.6 Types of Asphalt for Surface Treatments

Types of Construction	Asphalt Cements		Emulsified and Cutback Asphalts														
			Emulsified (Anionic)						Emulsified (Cationic)				Medium Curing (MC)			Slow Curing (SC)	
	AC-2.5[3]	AC-5[4]	RS-1	RS-2	MS-1	HFMS-1	SS-1	SS-1h	CRS-1	CRS-2	CSS-1	CSS-1h	30	70	250	70	250
Surface Treatments With Cover Aggregates	×	×	×	×	×	×			×	×							
Prime[5]							×[2]	×[2]			×[2]	×[2]	×	×	×		
Dust Laying/Road Oiling							×[1]	×[1]			×[1]	×[1]	×	×	×	×	×
Tack Coat							×[1]	×[1]			×[1]	×[1]					
Fog Seal							×[1]	×[1]			×[1]	×[1]					
Sand Seal			×		×	×			×								
Slurry Seal							×	×			×	×					

[1] Water diluted.
[2] Mixed in prime only.
[3] AR-1000 or 200/300 pen.
[4] AR-2000 or 120/150 pen.
[5] MS-2 or CMS-2 may also be used.

Sprayed Asphalt Surface Treatments

Sprayed asphalt treatments do not have cover aggregates. They are (a) prime coats, (b) dust-laying treatments, (c) road oiling treatments, (d) tack coats, and (e) fog seals.

Prime Coat

A prime coat is an application of low viscosity cutback or emulsified asphalt to a granular base in preparation for a subsequent asphalt course. In order for the prime coat to satisfy these criteria it needs to penetrate into the base course. The prime coat is designed to perform several functions:

- To coat and bond loose mineral particles on the surface of the base.
- To harden or toughen the surface.
- To waterproof the surface of the base.
- To plug capillary voids.
- To provide adhesion between the base and the next course.

The untreated base must be properly shaped and compacted prior to application of the prime coat. Loose material must be swept from the surface and the surface should be dry or only slightly damp.

Cutback asphalts normally used for priming are MC-30 and MC-70. However, MC-250 is sometimes used on loosely bonded, open textured surfaces. The ideal rate of application is the amount that will be completely absorbed by the surface in a 24-hour period. In general, this amount is between 0.90 and 2.25 litres/m^2 (0.20 and 0.50 gal/yd^2).

Slow setting emulsified asphalts can be used for mixed-in-priming. The quantity of emulsion to be used depends upon the nature of the granular base and weather conditions. The gradation of the aggregate, size of void spaces, and absorption of the aggregate all affect the rate of application. Generally, between 0.3 to 0.9 litre/m^2/25 mm (0.1 to 0.3 gal/yd^2/in.) SS-1, SS-1h, CSS-1 or CSS-1h would be mixed-in for this purpose. If an excess of asphalt residue is found on the surface when the emulsion has broken, a very light sand dusting will soak up the surplus material.

Just as for single and multiple surface treatments, building paper can be laid down at both the beginning and end of each run to create a neat starting and ending line.

In areas where the asphalt primer has not been completely absorbed by the base within the 24 hours, a sand blotter course might be applied. The amount applied should be sufficient to blot up the excess asphalt and to prevent it from picking up under traffic. All loose sand should be swept from the base prior to placing additional asphalt courses on the primed surface.

Temperatures for priming should be above 10° C (50° F) in the shade. This is a basic guide for most asphalt spray treatments.

Dust Laying

Dust laying consists of spraying an untreated surface with a diluted slow-setting asphalt emulsion or a low-viscosity cutback. The asphalt and diluent penetrate and coat the fine particles and temporarily relieve the dust nuisance.

Asphalt cutback is usually sprayed at the rate of 0.45 to 2.25 litres/m^2 (0.1 to 0.5 gal/yd^2). When emulsion is used, it should be diluted with up to five or more parts of water by volume. The dust stirred by traffic between applications eventually conglomerates and no longer rises. This is an effective treatment in very dusty areas where one application of cutback asphalt is insufficient.

Road Oiling

Road oiling differs from dust laying in that it is usually done as part of a planned buildup of low-cost road surfaces over several years. Each application may be mechanically mixed with the material being treated, or it may be allowed to penetrate. The light oils in the road oil penetrate into the granular subgrade and tend to repel moisture absorption. The objective in all road-oiling work is a firm dustless wearing course mat and a strong subgrade layer that will not become saturated with water.

Because soils vary widely, procedures for oiling are a matter for local trial and error rather than exact analysis. The amount of road oil required in the first year's work will vary from 3.40 to 4.50 litres/m^2 (0.75 to 1.0 gal/yd^2). This should be applied in three applications. The first application is applied at the rate of about one-half the total; succeeding applications are made in two equal, smaller amounts.

Road-oil treatments are placed several weeks apart, depending upon the character of the asphalt soil mat.

If some breakup occurs after the first winter, light scarifying and reworking the second year will produce a thicker and stronger surface.

Tack Coat

A tack coat is a very light spray application of diluted asphalt emulsion. It is used to ensure a bond between a surface being paved and the new course. For most overlays, a tack coat is advisable. Perhaps the only exception is when an additional course is placed within one or two days on a freshly-laid asphalt surface that has not been turned-over to traffic. In this case, ample bond between the two courses should develop without the use of a tack coat. In all cases, however, the surface must be clean and free of dust and loose material.

The more common emulsion types for tack coats are diluted SS-1, SS-1h, CSS-1, and CSS-1h. The emulsion is diluted by adding an equal amount of water. First, a test dilution is made to be certain that the water to be used is compatible with the emulsion. To prevent premature breaking, the water is always added to the emulsion, not the emulsion to the water. The diluted material is then applied at a rate of 0.25 to 0.70 litre/m^2 (0.05 to 0.15 gal/yd^2). No more tack coat should be applied to an area than can be covered by the same day's operations.

Tack coats should not be applied during periods of cold or wet weather. Best results are obtained if the road surface is dry, has a surface temperature above 27° C (80° F), and there is no threat of rain.

The goal is a very thin but uniform coating of asphalt left on the surface when the emulsion has broken. Too much tack coat may create a plane of slippage between the two pavement courses as the asphalt acts as a lubricant rather than an adhesive. It may even create "fat spots" or bleeding on the surface of the new pavement, a condition that is not only unsightly, but produces a dangerously slick pavement. Pneumatic-tired rolling of spotty tack coats will help spread the asphalt for better coverage. It will also help to lessen the probability of fat spots. After spraying the tack coat, enough time must be allowed for complete breaking to occur before the overlay is placed. Traffic should be kept off the tacked area.

A tack coat is also an essential part of a good patching operation. First, the area to be patched must be thoroughly cleaned and all loose material removed. Then a fairly heavy tack coat of asphalt emulsion is sprayed, or painted, over the entire area, including the vertical sides. The tack helps hold the patch in place and imparts a watertight seal between the patch and the surrounding pavement.

Fog Seal

A fog seal is a light application to an existing surface of a slow-setting asphalt emulsion diluted with water, similar to a tack coat. It can be diluted in varying proportions up to one part emulsion to five parts water, but in most cases a one to one dilution is used. Grades of asphalt emulsion normally used for this purpose are SS-1, SS-1h, CSS-1, or CSS-1h.

A fog seal can be a valuable maintenance aid when used for its intended purpose. It is not a substitute for an asphalt-aggregate surface treatment or a slurry seal. It is used to renew old asphalt surfaces that have become dry and embrittled with age, to seal small cracks and surface voids and to inhibit raveling. The fairly low viscosity diluted emulsion flows easily into the cracks and surface voids. It also coats aggregate particles on the surface. This corrective action will prolong pavement life and may delay the time when major maintenance or rehabilitation is needed.

The total quantity of fog seal used is normally in the order of 0.45 to 0.70 litre/m^2 (0.1 to 0.15 gal/yd^2) of diluted material. Exact quantities are determined by the surface texture, dryness, and degree of cracking or raveling of the pavement on which the fog seal is sprayed. Over-application must be avoided, as this would result in an asphalt pickup by vehicles and possibly a slippery surface. If an excess of emulsion is applied, a light dusting of the affected area with a fine sand may remedy the problem.

Traffic must be kept off the fog seal until the emulsion breaks and is substantially absorbed into the existing surface. This curing period may range from one hour for hot, dry conditions to as much as 3 hours or longer for cool, humid conditions. The curing period can often be reduced by rolling with a pneumatic-tired roller.

Emulsified Asphalt Slurry Seal

Description

A slurry seal is a designed mixture of well-graded fine aggregate, mineral filler (if needed), emulsified asphalt, and water applied to a pavement as a surface treatment. It is used in both the preventive and corrective maintenance of asphalt pavement surfaces. It does not increase the structural strength of a pavement section. Any pavement that is structurally weak in localized areas should be repaired before applying the slurry seal. All ruts, humps, low pavement edges, crown deficiencies, waves, or other surface irregularities that diminish the riding quality should be corrected before placing the slurry seal.

Slurry seal, when properly applied to the surface of an older pavement, can be used quite effectively. Its timely application will help reduce surface distress caused by oxidation of the asphalt and embrittlement of the paving mixture. It will seal the surface cracks, stop raveling and loss of matrix, make open surfaces impermeable to air and water, and improve skid resistance and pavement appearance.

Slurry seal has a number of advantages; some are listed below:

- Rapid application
- No loose cover aggregate
- Excellent surface texture for paint striping
- Ability to correct minor surface irregularities
- Minimum loss of curb height
- No need for manhole or other structure adjustments
- Improves the aesthetics of the existing pavement
- In many cases, the relatively low cost of the treatment makes it practical to import aggregates for special effects, such as high skid resistance, color contrast and noise reduction

Slurry Seal Machine

The slurry is usually applied in a thickness of 3 to 6 mm (⅛ to ¼ in.). It comes directly from a traveling mixing plant into an attached spreader box that spreads the slurry by a squeegee-type action (Figure 8.11). The machine used for production of the slurry seal is a self-contained, continuous-flow mixing unit. It is capable of delivering accurately to the mixing chamber predetermined amounts of aggregate, mineral filler (if required), water, and asphalt emulsion. It also discharges the thoroughly mixed materials onto the prepared surface. Certain basic features are common to all batch type slurry machines. They are truck-mounted units with separate storage tanks, bins, and metering systems for emulsified asphalt, water, aggregate and mineral filler. The slurry machine has a continuous-flow mixing unit, either single or double pugmill, from which the slurry is discharged into a spreader box. The box is equipped with flexible squeegees and a device for adjustable width. Spreader boxes may be equipped with hydraulically-powered augers to keep the slurry in motion and help keep the mixture uniformly spread across the spreader box width. These are helpful when quick-set (QS) emulsion is used. One type of slurry mixer unit is depicted in the schematic drawing, Figure 8.12. Continuously self-loading machines capable of placing many kilometers (miles) per day of coarse slurry are in use in many regions of the United States.

Materials

The aggregate used in slurry seal must be clean, angular, durable, well graded, and uniform. An individual aggregate or a blend of aggregates to be used in a slurry mix should meet these limits:

- Sand equivalent value, ASTM D 2419 (AASHTO T 176) = 45 minimum.
- Los Angeles abrasion loss, ASTM C 131 (AASHTO T 96) Grading C or D = 35 maximum.

Also, the amount of smooth-textured sand of less than 1.25 percent water absorption is limited to not more than 50 percent of the total combined aggregate.

The three generally accepted gradings used for slurry mixtures are shown in Table 8.7.

Type I is used for maximum crack penetration. Also, it makes an excellent pretreatment for hot-mix overlay or chip seal. It is usually used in low density traffic areas such as light aircraft airfields, parking areas, or shoulders where the primary objective is sealing.

Type II is the most widely used gradation. It is used to seal; to correct severe raveling, oxidation, and loss of matrix; and to improve skid resistance. It is used for moderate traffic, depending upon the quality of aggregates available and the design.

Type III is used to correct surface conditions, as the first course in multicourse applications for heavier traffic, and to import skid resistance.

Emulsified asphalt used in the slurry mix may be SS-1, SS-1h, CS-1, or CSS-1h. Quick-setting (QS) asphalt emulsion is being used when early opening to traffic is necessary. Sometimes, a small amount of liquid or powdered additive is added to the asphalt emulsion to control the setting time of the slurry mixture. This additive starts the set in anionic quick-set emulsions. It retards the set in cationic quick-set emulsions.

It is almost always necessary to add a small amount of mineral filler—hydrated lime, limestone dust, portland cement, or fly ash—to aid in stabilizing and setting the slurry.

Water used in the slurry should be potable and compatible with the mix.

For slurry seal mix design the following sources are recommended:

ASTM D 3910 "Standard Practices for Design, Testing and Construction of Slurry Seal"

American Society for Testing and Materials
1916 Race Street
Philadelphia, PA 19103

"Recommended Performance Guidelines for Emulsified Asphalt Slurry Seal Surfaces" A105

International Slurry Surfacing Association
1101 Connecticut Avenue
Washington, D.C. 20036

"Recommended Guideline for Slurry Seal"
Asphalt Emulsion Manufacturers Association
#3 Church Circle
Annapolis, MD 21401

Preparing the Mixture

Blending the slurry seal materials in varying proportions in the laboratory is a great aid in selecting the proper mixture. Correct blending should produce a slurry with a creamy texture that will flow smoothly in a rolling wave ahead of the strike-off squeegee. This slurry should be a semifluid, homogeneous mass with no emulsion runoff.

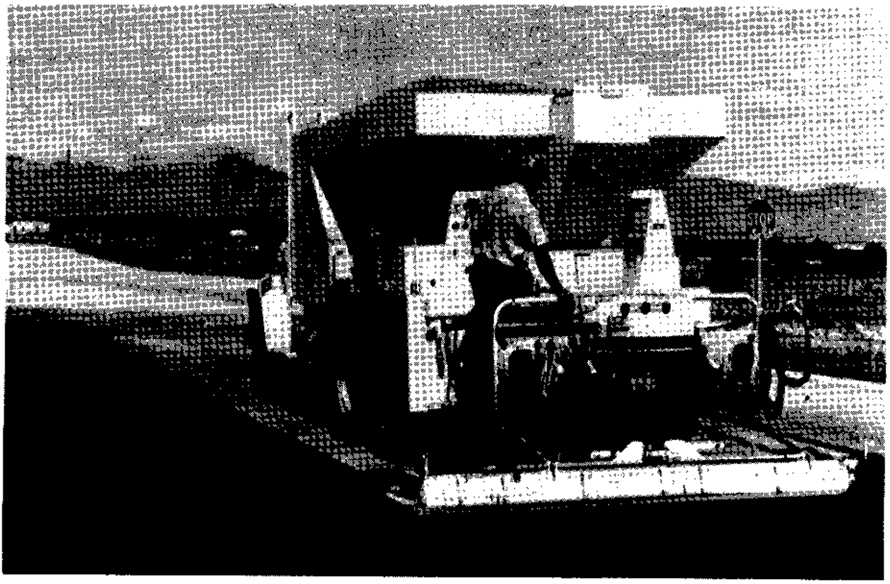


Figure 8.11 Slurry Seal Machine. *(Courtesy ScanRoad Incorporated)*

After mix proportions have been determined in the laboratory, it is always advisable to place one or more trial mixes. This should be done either at the job site or in a location where small spreads of the slurry seal would not be objectionable. The trial sections serve a two-fold purpose. First, to calibrate the feeding and metering devices on the slurry machine. Aggregate flow should be determined for different gate openings and the amount of emulsion pumped per revolution of the aggregate feed belt. Second, to see if the slurry mix proportions are right. It is often necessary to make several trial runs to find the best blend of materials—even when starting with a laboratory-determined mixture. Always use exactly the same materials in the laboratory design work that are to be used for the field construction.

If lumping, balling, or unmixed aggregate is observed, the slurry should be removed from the pavement. It should also be removed if coarser

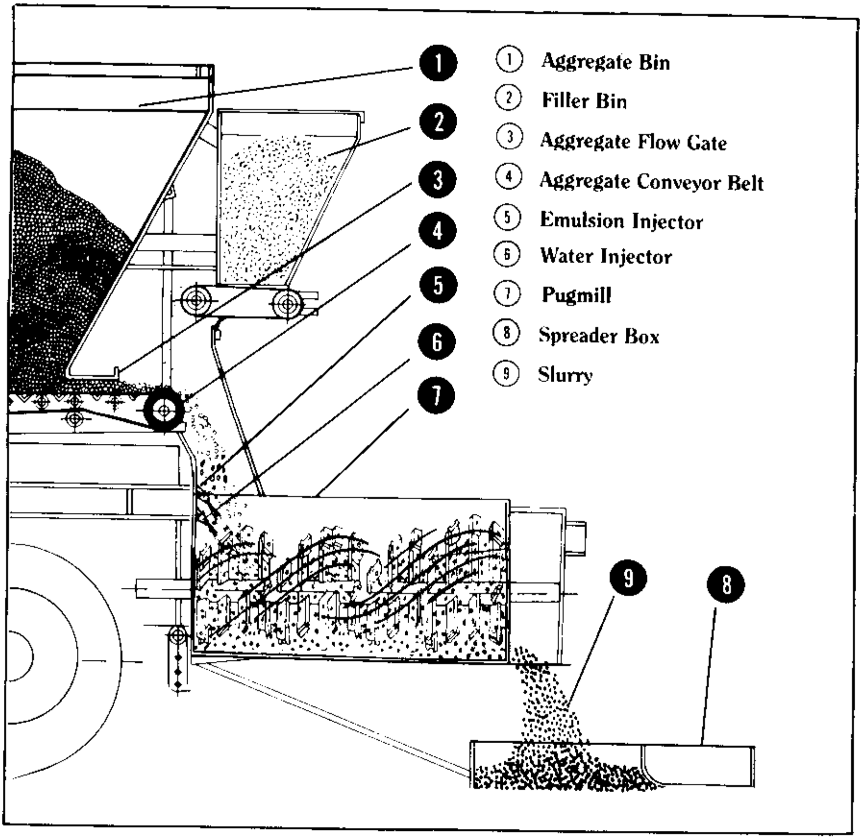


Figure 8.12 **Flow Diagram of a Typical Slurry Seal Mixer.** *(Courtesy ScanRoad, Inc.)*

aggregate particles settle to the bottom of the mix. Streaks, such as those caused by oversized aggregate, should be repaired at once with a hand squeegee.

Placing the Mixture

Just before applying the slurry, the pavement surface should be cleaned of all dirt, dust, mud spots, vegetation and other foreign matter. A tack coat of diluted emulsified asphalt of the same type and grade specified for the slurry may be required directly ahead of the slurry application. With relatively new asphalt pavements, the tack coat may be omitted. The pavement surface should always be damp but with no free water in front of the slurry machine.

It is especially important to get a good homogenous mix, one that will produce a slurry with a creamy texture that will flow smoothly in a rollaway wave inside the spreader box. A non-homogenous mixture will cause an asphalt rich surface and many ensuing problems.

Table 8.7 Slurry Mixture Gradings*

Type of Slurry	I	II	III
General Usage	Crack filling & fine seal	General seal, medium textured surfaces	1st and or 2nd application, two-course slurry, highly textured surfaces
Sieve Size	Percent Passing		
9.5mm (⅜ in.)	100	100	100
4.75mm (No. 4)	100	90-100	70-90
2.36mm (No. 8)	90-100	65-90	45-70
1.18mm (No. 16)	65-90	45-70	28-50
600 µm (No. 30)	40-65	30-50	19-34
300 µm (No. 50)	25-42	18-30	12-25
150 µm (No. 100)	15-30	10-21	7-18
75 µm (No. 200)	10-20	5-15	5-15
Residual Asphalt Content, % Weight of dry aggregate	10-16	7.5-13.5	6.5-12
Application Rate, kg/m² (lb/yd²), based on mass (weight) of dry aggregate	3-5.5 (6-10)	5.5-8 (10-15)	8 (15) or more

Recommended by International Slurry Surfacing Association.

Special care must be taken with longitudinal and transverse joints to prevent excessive buildup of slurry (ridging) and to prevent streaking. It is best to make the joint after the first placed lane is either completely cured or is still in a semi-fluid condition. For good appearance and durability, a joint should not be made when the lane to be joined is only partially set, as tearing and scarring may result.

Quite often a canvas or burlap drag is pulled behind the spreader box to improve the joint and overall surface appearance. Drags should be changed regularly. Hand squeegees and hand drags are used to improve joints and place the slurry in areas inaccessible to the machine.

On flat grades, slurry in the spreader box must be kept at a uniform thickness. On high crowned pavements or superelevated curves, slurry should be diverted to the high side of the spreader box. Gravity will keep the low side filled. Spreading slurry in hilly areas is easier if the slurry machine travels in an uphill direction. If circumstances require placing in a downhill direction, the slurry must be thickened to contain it from flowing ahead of the machine.

Rolling a slurry seal is only needed in those areas where pneumatic-tired rolling will improve durability. Such areas include taxiways, runways, truck terminal yards, and intersections of heavily traveled roads. All of these are subject to power steering turns, braking, or acceleration forces. For rolling, a 4.5 tonne (5 ton) pneumatic roller with 345 kPa (50 psi) tire pressure will be most effective. Rolling can start as soon as clear water can be pressed out of the slurry mixture with a piece of paper without discoloring the paper. In most cases, traffic will iron out the slurry and close any hairline cracks of dehydration. Rolling usually is not needed unless the thickness is more than 6mm (¼ in.) or unless late season work is involved.

Curing/Traffic Control

Slurry should be placed only when the temperature is at least 10° C (50° F) and rising and when no rain is expected. A newly placed slurry should not be opened to traffic until it has completely cured. As with rolling, traffic generally can be allowed on the slurry as soon as clear water can be pressed out of the slurry mixture with a piece of paper without discoloring the paper. The traffic, of course, must be controlled somewhat, as quick stops or accelerations and the turning of wheels while parked will cause damage to the slurry.

Chapter 9
Asphalt in Pavement Maintenance and Rehabilitation

This chapter deals with pavement maintenance and rehabilitation. It provides useful and practical information about methods, and terminology that apply to the use of asphalt in the maintenance and rehabilitation of all types of pavement structures.

Large differences in soil types, climate, traffic and other factors make for greatly varying problems, even within small areas. Some regions are rugged and mountainous while others are fairly smooth and level; some have heavy rainfall, others are semi-arid; some highways and streets must accommodate vehicles carrying coal, ore, logs, or other heavy loads, while others are subjected to only light-weight traffic.

Yet, despite these differences, there are maintenance and rehabilitation methods that can be used equally well in all regions. Presenting some of these methods is the purpose of this chapter.

SECTION 9.1 Planning for Improvement/Pavement Management
☐ Introduction
☐ Pavement Management
☐ Strategies For Future Improvements
☐ Definitions Relating to Maintenance and Rehabilitation

SECTION 9.2 Maintenance of Asphalt Pavements
☐ Introduction
☐ Patching
☐ Causes and Repair of Cracks
☐ Causes and Repair of Distortions
☐ Causes and Repair of Slippery Surfaces
☐ Surface Treatment Problems

SECTION 9.3 Asphalt in Rigid Pavement Maintenance

- ☐ Introduction
- ☐ Joint and Crack Maintenance
- ☐ Causes and Repair of Distortion
- ☐ Causes and Repair of Disintegration
- ☐ Eliminating Skid Hazards

SECTION 9.4 Asphalt in Pavement Rehabilitation

- ☐ Introduction
- ☐ Evaluating Pavement Condition
- ☐ Asphalt Overlays

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9.1 Planning for Improvement/ Pavement Management

Introduction

Anything built with construction materials will deteriorate with the passage of time. Roads, streets, and parking facilities are no exception. Roads deteriorate relatively slowly during the early years of their design life, but as they approach the end of their design life, this rate accelerates. Proper maintenance will significantly lengthen the life of a pavement system, but eventually rehabilitation and reconstruction is required. In many cases, more than one technique or combination of techniques can be used to solve a specific problem. The techniques selected should be based on a thorough engineering analysis of the pavement's proposed application and an economic analysis of costs and benefits.

Pavement Management

The key to good management is careful planning and programming of maintenance and rehabilitation. Pavement management systems aid in the planning and programming of work. Pavement management is defined as those activities associated with the operation of a roadway system such as planning, data collection, design, construction and maintenance or rehabilitation.

A pavement management system is a decision system to integrate pavement management activities with roadway evaluation and possible computer simulation to achieve the best use of available funds by comparing investment alternatives and coordinating design, construction and maintenance.

Pavement management systems vary considerably in their scope and complexity. Large organizations such as state highway departments or large cities may develop a system which contains complex predictive models custom designed for their situation. Smaller organizations frequently use off the shelf systems with slight modifications. Both large and small pavement management systems generally have the following parts: prediction, strategies, implementation and evaluation.

Inventory data provides a data base to supply information for decisions. Inventory should contain data of the network layout, construction history, traffic data, ride quality, surface condition and structural adequacy. Structural and geometric roadway evaluation is used to update the database.

Structural and geometric roadway evaluations should be done to all roads in the system. Geometric evaluation may be done with construction plans, field inspection or photographic inventory techniques. Surface condition is usually evaluated by field inspection but can be evaluated with photographic techniques. Structural adequacy may be evaluated using surface condition or measuring pavement deflection.

Figure 9.1 shows a simple rehabilitation/maintenance guide. The steps are indicated that can be used to formulate a plan. Although more sophisticated planning systems are available, this chart provides a framework for engineering maintenance and rehabilitation decisions.

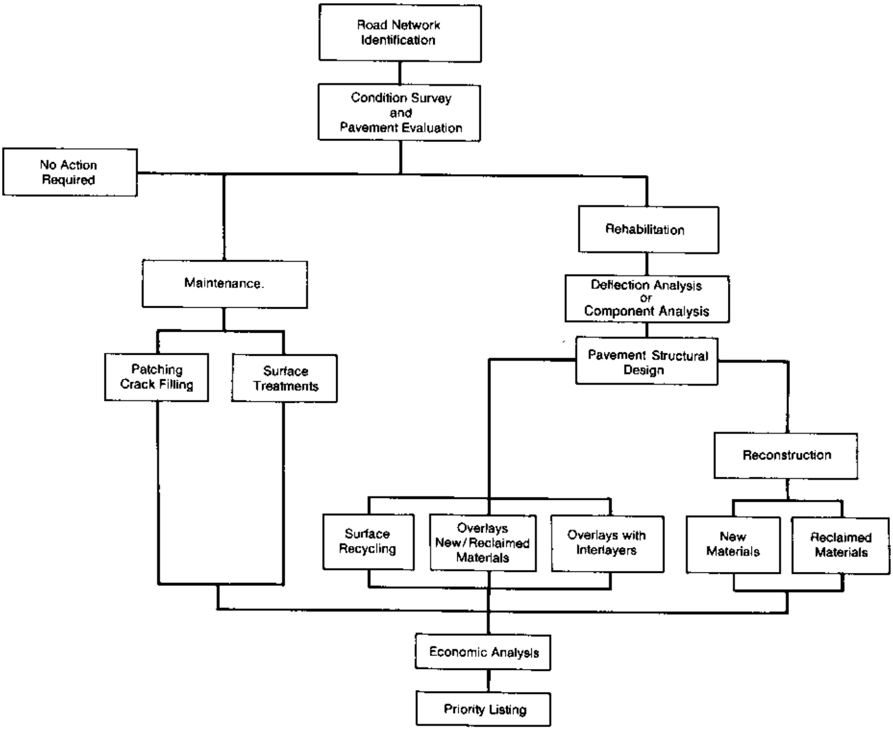
Road Network Identification
Condition Survey and Pavement Evaluation
No Action Required
Maintenance.
Patching Crack Filling
Surface Treatments
Rehabilitation
Deflection Analysis or Component Analysis
Pavement Structural Design
Reconstruction
Surface Recycling
Overlays New/Reclaimed Materials
Overlays with Interlayers
New Materials
Reclaimed Materials
Economic Analysis
Priority Listing

Figure 9.1 Plan-Development Guide.

The first step in the development of a program is to identify pavement sections. The sections should be distinguished by location, pavement type, thickness of each course, type of base, type of subbase (if any), and an index that relates to the support strength of the subgrade material. Any change in these items may require that the section be redesignated because it will react to traffic and environment in a different manner.

The second step is to conduct a thorough condition survey, recording the physical condition of each pavement section. It is important that this survey be conducted by a qualified individual. All problems on the roadway and paved shoulder must be noted and the probable causes determined. In many cases the causes and solutions will be readily evident to a qualified inspector.

This chapter contains general guidelines related to causes and solutions in pavement problems.

At this point a determination is made regarding the level of maintenance required. If maintenance is called for, the type of maintenance should be determined. Patching and crack filling or surface treatments should be considered. If the condition survey indicates that more than maintenance is necessary, or that the problem may be the result of overloading, a deflection analysis should be conducted. If a deflection analysis is not practical, a component analysis can be used. Procedures for conducting and evaluating the results of a deflection analysis or a component analysis are contained in the Asphalt Institute publication Asphalt Overlays for Highway and Street Rehabilitation (MS-17). See also information in Chapter 11, Section 11.7. The analysis will provide data that can be used to develop appropriate corrective designs, taking into account anticipated changes in future use. Table 9.1 contains various maintenance and rehabilitation alternatives to be considered when developing pavement strategies.

The last step is to develop an economic analysis and a priority listing of the maintenance, rehabilitation, or reconstruction required. This information may then be used to develop a time-based plan that takes into consideration the amount of funds available. Until this point, the actual amount of money available should not be a consideration. If prior consideration is given to funds available, judgment may be clouded and improper engineering decisions result. Delaying necessary maintenance almost always results in greater expenditures when the maintenance is finally accomplished. Timely maintenance is an important element of a good pavement management system.

Due to budget limitations, however, it is not always possible to accomplish the desired level of maintenance. Stop-gap procedures are then used until sufficient funding becomes available. A clear understanding of the limitations and proper use of each type of maintenance treatment will enable an engineer to make effective use his resources. But, it should be remembered that deferred maintenance costs more. This concept is illustrated by the example in Figure 9.2.

Strategies for Future Improvements

The predictive capabilities of a pavement management system allow an estimation of future condition. A system may have very simple predictive models or it may have many complex interwoven models relating traffic, environment, construction materials, surface distresses and deflection. Typically a system will contain models which relate age or traffic to surface condition, ride quality and structural adequacy for classes of roadway grouped by traffic, environmental zone and pavement thickness.

A sound pavement management system which provides an adequate plan for maintenance and rehabilitation must consider all costs of pavements.

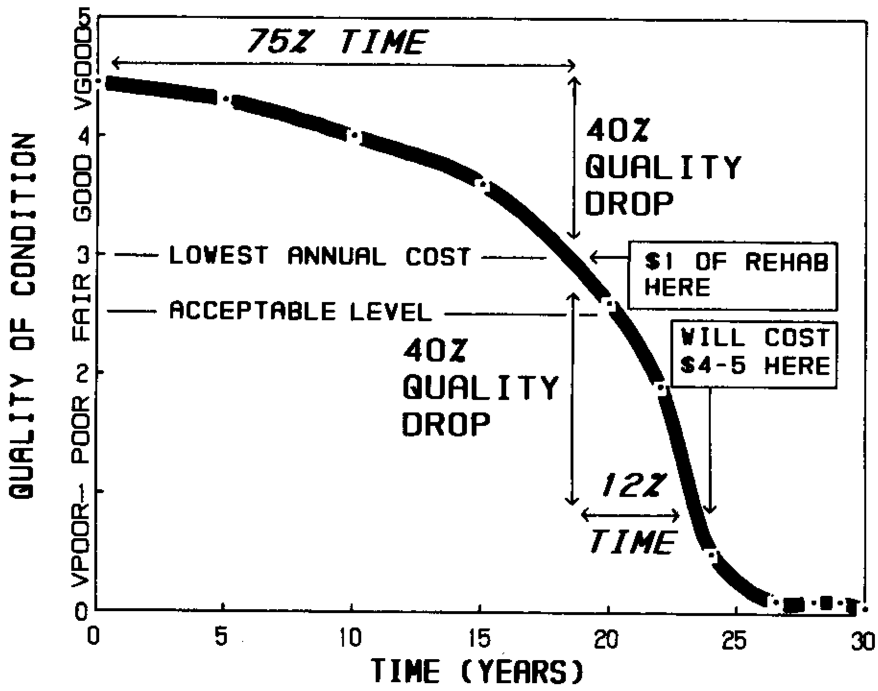

Figure 9.2 Example of Road Deterioration vs Time.

Maintenance costs should include routine maintenance and major planned maintenance. User delay cost while under construction and the cost of denied access to abutting property owners should also be included.

Using pavement management systems an agency can evaluate the application of various maintenance and rehabilitation strategies. The strategy chosen as routine policy, seal coats for example, should be selected based on economic soundness. Using a pavement management system, alternate strategies can be simulated and their effects on ride, surface condition and structural adequacy can be predicted. For example the system can simulate the application of seal coats, thin overlays and thick overlays. Costs, surface condition and benefits are predicted into the future and an economic analysis can be done to determine which policy is most economical. In addition to which treatment is economically the best, a pavement management system can evaluate treatment application for different levels of distress. The selection of different surface condition requirements will change future estimates of surface condition and ride quality. The resulting costs and benefits associated with each selected condition requirement can be evaluated to determine the best time of treatment application.

The predictive capabilities of a pavement management system allow an agency to choose cost effective treatments and condition requirements for various classes of roadway in their system. It also allows the agency to predict the effects of deferred rehabilitation or major maintenance.

Pavement structures suffer accelerated deterioration as major maintenance and rehabilitation are deferred. A pavement management system allows the agency to predict the effects of reduced budgets on average system condition and to estimate the cost of returning their roadway system to a specified level of service. With a pavement management system an agency has the ability to select a program which will meet specified criteria such as minimizing costs to meet specific service levels while maximizing benefits. If budgets are curtailed or increased an agency has the ability to estimate the effects of the budget change or user benefits and future rehabilitation demand and to determine the program changes required to minimize the effects of reduced budgets or to maximize the effects of increased budgets.

Definitions Relating to Maintenance and Rehabilitation

Each locality has a number of unique terms relating to pavement maintenance and rehabilitation. The following terms, however, are a compilation of some of the more basic and widely used definitions.

In the United States, because of rules governing the financing of highway and street improvements, there is an important distinction between maintenance and other pavement rehabilitation techniques, including overlays. In some jurisdictions overlays less than 25mm (1 in.) thick, for example, are considered maintenance, and overlays of greater thickness are considered capital improvements. The U.S. Federal Highway Administration defines maintenance as including "work primarily for rejuvenation or protection of existing surfaces; resurfacing of less than 19mm (¾-in.) minimum thickness or of short length; patching and repair of minor failures; and undersealing of concrete slabs other than essential as a part of restoration for resurfacing."

Regardless of definition, an overlay can be as thin as 25mm (1 in.) to improve the structural strength of asphalt pavements. Thinner overlays generally are used to improve rideability, waterproof the pavement surface, or extend surface life without having been "designed," in the sense used here. For portland cement concrete pavements the Institute recommends that asphalt concrete overlays be at least 100mm (4.0 in.) thick to minimize reflection cracking. Thicker overlays may be required, as indicated by the design procedures.

Definitions

Alligator Cracks—Interconnected cracks forming a series of small blocks resembling an alligator's skin or chicken wire.

Asphalt Concrete—High-quality, thoroughly-controlled hot mixture of asphalt cement and well-graded, high quality aggregate, thoroughly compacted into a uniform dense mass.

Asphalt Leveling Course—A course (asphalt-aggregate mixture) of variable thickness used to eliminate irregularities in the contour of an existing surface prior to placement of an overlay.

Asphalt Overlay—One or more courses of asphalt construction on an existing pavement. The overlay generally includes a leveling course, to correct the contour of the old pavement, followed by uniform course or courses to provide needed thickness.

Asphalt Pavements—Pavements consisting of a surface course of mineral aggregate coated and cemented together with asphalt cement on supporting courses such as asphalt bases; crushed stone, slag, or gravel; or on portland cement concrete, brick, or block pavement.

Bleeding or Flushing Asphalt—The upward movement of asphalt in an asphalt pavement resulting in the formation of a film of asphalt on the surface.

Blow-Up—The localized buckling or shattering of a rigid type pavement, occurring usually at a transverse crack or joint.

Channels (Ruts)—Channelized depressions which may develop in the wheel tracks of an asphalt pavement.

City Street—An urban street whose traffic is predominantly local in character.

Corrugations (Washboarding)—A form of plastic movement typified by ripples across the pavement surface.

Crack—Separation of the pavement due to natural causes, traffic action, or reflections from an underlying pavement.

Deflection—The amount of downward vertical movement of a surface due to the application of a load to the surface.

Rebound Deflection—The amount of vertical rebound of a surface that occurs when a load is removed from the surface.

Representative Rebound Deflection—The mean value of measured rebound deflections in a test section plus two standard deviations, adjusted for temperature and most critical period of the year for pavement performance.

Residual Deflection—The difference between original and final elevations of a surface resulting from the application to, and removal of one or more loads from, the surface.

Design Period—The number of years from the initial application of traffic until the first planned major resurfacing or overlay. This term should not be confused with pavement life or Analysis Period. By adding asphalt overlays as required, pavement life may be extended indefinitely, or until geometric considerations or other factors make the pavement obsolete.

Disintegration—The breaking up of a pavement into small, loose fragments due to traffic, weathering, improper mix design or poor construction.

Distortion—Any change of a pavement surface from its original shape.

Fault—A difference in elevation of two slabs at a joint or crack.

Grade Depressions—Localized low areas of limited size which may or may not be accompanied by cracking.

Hydroplaning—The dangerous action of a vehicle being driven on a pavement over which a film of rain or other water has formed; on reaching a certain speed, the vehicle's tires tend to ride upon the water surface rather than the pavement, drastically reducing the driver's control of the vehicle.

Instability—The lack of internal resistance tending to cause movement or distortion of a pavement structure.

Longitudinal Crack—A crack that follows a course approximately parallel to the centerline.

Polished Aggregate—Aggregate particles in a pavement surface with edges that have been rounded and surfaces polished smooth by traffic.

Potholes—Bowl-shaped holes of varying sizes in the pavement, resulting from localized disintegration.

Present Serviceability—The ability of a specific section of pavement to serve the intended traffic usage.

Present Serviceability Index (PSI)—A mathematical combination of values, obtained from certain physical measurements of a large number of pavements, so formulated as to predict, within prescribed limits, the Present Serviceability Rating (PSR) for those pavements.

Present Serviceability Rating (PSR)—The mean of the individual ratings made by the members of a specific panel selected for the purpose.

Pumping or Blowing—Slab movement under passing loads resulting in the ejection of mixtures of water, sand, clay, and/or silt along transverse or longitudinal joints and cracks, and along pavement edges.

Raveling—The progressive separation of aggregate particles in a pavement from the surface downward or from the edges inward.

Reflection Cracks—Cracks in asphalt overlays that reflect the crack pattern in the pavement structure underneath.

Roughometer—A single-wheeled trailer instrumented to measure the roughness of a pavement surface in accumulated millimetres (inches) per mile.

Rut—(Refer to Channels).

Scaling—The peeling away or disintegrating of the surface of portland cement concrete.

Shoving—A form of plastic movement resulting in localized bulging of the pavement.

Skid Hazard—Any condition that might contribute to making a pavement slippery.

Slippage Cracks—Cracks, sometimes crescent-shaped, that point in the direction of the thrust of the wheels on the pavement surface.

Spalling—The breaking or chipping of the pavement at joints, cracks, or edges, usually resulting in fragments with feather edges.

Transverse Cracks—A crack that follows a course approximately at right angles to the centerline.

Upheaval—The localized upward displacement of a pavement due to swelling of the subgrade or some portion of the pavement structure.

	POSSIBLE CAUSE				MAINTENANCE^1				REHABILITATION^2				RECONSTRUCTION^3	
PROBLEM	Structural Failure	Mix Composition	Temp. or Moisture Changes	Const.	Patching & Routine Maintenance	Fog Seal	Surface Treatment	Slurry Seal	Surface Recycling	Thin Overlay	Open-Graded Surface	Structural^8 Overlay	Recycled Materials	New Materials
Alligator Cracking	X				X^4		X^5	X^5				X	X	X
Edge Joint Cracks	X		X	X	X									
Reflection Cracks			X		X		X^5	X^5				X		
Shrinkage Cracking		X	X				X	X	X			X		
Slippage Cracks				X	X									
Rutting	X	X		X					X	X^6		X	X	X
Corrugation	X	X		X	X^4				X	X^7		X	X	X
Depressions	X			X	X								X	X
Upheaval			X		X								X	X
Potholes	X	X	X	X	X							X		
Raveling		X		X		X^5	X	X	X	X	X			
Flushing Asphalt		X		X			X^6		X		X			
Polished Aggregate		X					X	X	X	X	X			
Loss Of Cover Aggregate		X		X			X^6			X	X			

NOTES: 1. Refer to Asphalt in Pavement Maintenance (MS-16), Asphalt Institute, for details. 2. When distress exceeds 40 percent of the surface area of the pavement. 3. If problem is extensive enough. 4. Deep patch-permanent repair. 5. Temporary repair. 6. When distress is minor. 7. Over planed surface 8. After propor surface preparation.

Table 9.1 Some Alternatives in Pavement Maintenance and Rehabilitation

9.2 Maintenance of Asphalt Pavements

Introduction

There are about two million miles of paved roads in the United States, all requiring maintenance. Original investment in these structures was in the billions of dollars. Their maintenance now costs millions of tax dollars annually. Obviously, timely and proper maintenance is imperative.

Maintenance Defined

Pavement maintenance is not easy to define. Highway departments agree in general as to what it is but there are some minor differences, chiefly in scope. Some call any pavement improvement "maintenance." Others include only the work which keeps the pavement in its as-constructed condition.

Taking all of these into consideration the definition which seems most nearly to fit is:

> Pavement maintenance is the routine work performed to keep a pavement, under normal conditions of traffic and normal forces of nature, as nearly as possible in its as-constructed condition.

Pavement, if maintained in its as-constructed condition, would theoretically last forever. But in practice, maintenance merely helps pavement perform satisfactorily during its designed life. It should be accomplished with minimum expense and interruption to traffic.

Maintenance not only preserves the pavement surface, but prevents accelerated wear. It should not be regarded as a temporary expedient, but as an investment in the pavement structure and insurance against costly pavement renovation.

Maintenance can be divided into three categories.

- Preventive Maintenance—that work which is done to prevent deterioration of a pavement, thus reducing the need for more substantial maintenance work (drainage maintenance, fog seals, etc.).
- Routine Maintenance—the day-to-day work that is necessary to preserve and keep a pavement as close to an as-constructed condition as possible (crack sealing, pothole patching, etc.).
- Major Maintenance—that work which is needed to restore a pavement to its as-constructed condition. It includes surface treatments, slurry seals, surface recycling, and thin overlays of less than 25 mm (1 in.) (some agencies consider these thin overlays as rehabilitation).

Importance of Frequent Inspection

Stresses producing minor defects are constantly working in all pavements. Such stresses may be caused by a change in temperature or moisture content, by traffic, or by small movements in the underlying or adjacent soil. Cracks, holes, depressions, and other types of distress are the visible evidence of pavement wear.

Early detection and repair of minor defects is, without doubt, the most important work done by maintenance crews. Cracks and other surface breaks, which in their first stages are almost unnoticeable, may develop into serious defects if not soon repaired. For this reason, frequent inspections of the pavement should be made by qualified technicians.

Upon detection of pavement distress, repairs should then be made as quickly as possible.

It is essential that the cause of pavement distresses be determined prior to efforts to remedy the distress or failure. In most instances, if the cause of a distress is not determined and corrected prior to repairs, the repair will suffer the same short life as did the original pavement. Sometimes the cause may be such that correction is not presently feasible and only remedial measures are possible. Regardless, the cause and effect should always be established.

Maintenance Procedures

Maintenance procedures for correcting distresses in asphalt pavements include patching, crack and surface sealing, and in some cases resurfacing. Patching, described in detail later, may be either a temporary or permanent repair. Crack sealing is accomplished using emulsified or cutback asphalts, special asphalt compounds, specialty crack and joint sealers or possibly sealing the entire surface area. Surface treatments with or without aggregate, and thin overlays can also be part of the maintenance procedure.

Asphalt Mixes and Materials

Asphalt concrete and other hot mixes make long-lasting patches that develop stability rapidly. They should be used whenever it is practical and economical. Other maintenance materials are cold-plant and road-mix asphalt mixtures employing medium-curing or emulsified asphalt for immediate use or for short-time stockpiling. Slow-curing cutback asphalt or emulsion containing solvent may be used for stockpiled patching mixes.

Lighter grades of rapid- and medium-curing cutback asphalts are used for sealing small cracks, as are asphalt emulsions and emulsion slurry seals. For large cracks, the same asphalts may be combined with fine aggregate or sand and broomed into place. These large cracks may then be sealed with asphalt and dusted with fine aggregate or mineral dust to prevent tracking. Modified asphalt and specially prepared crack sealers are also used extensively.

Patching

Patching is probably the most widespread method of repair in highway and street maintenance. All pavements require patching at one time or another. If holes do not occur from natural causes, man-made service cuts and trenches produce them. Defects vary from cracked areas and shallow abrasions to deep holes.

Patching requires skill and close supervision. Prompt repair of small breaks will help keep down expenditures because once an area is broken and water enters the subgrade, a large failure will result.

Deep Patches

Deep patches at least 100 mm (4 in.) in depth, are used for making permanent repairs to the pavement. The material in the area to be repaired should be removed to the depth necessary to reach firm support. This may mean removing some of the subgrade. The excavation should also extend at least 0.3 metre (a foot) into the good pavement surrounding the area to be patched. Holes should be square-edged and the cuts rectangular in shape (Figure 9.3). A pavement saw makes a fast and clean cut. The vertical faces of the hole are then given a tack coat (Figure 9.4).

If the bottom of the hole is a granular base that is part of the pavement structure, it should be primed before the asphalt patching mix is placed. If the depth of the hole extends to or into the subgrade and a full-depth asphalt patch is contemplated, a prime coat is not necessary.

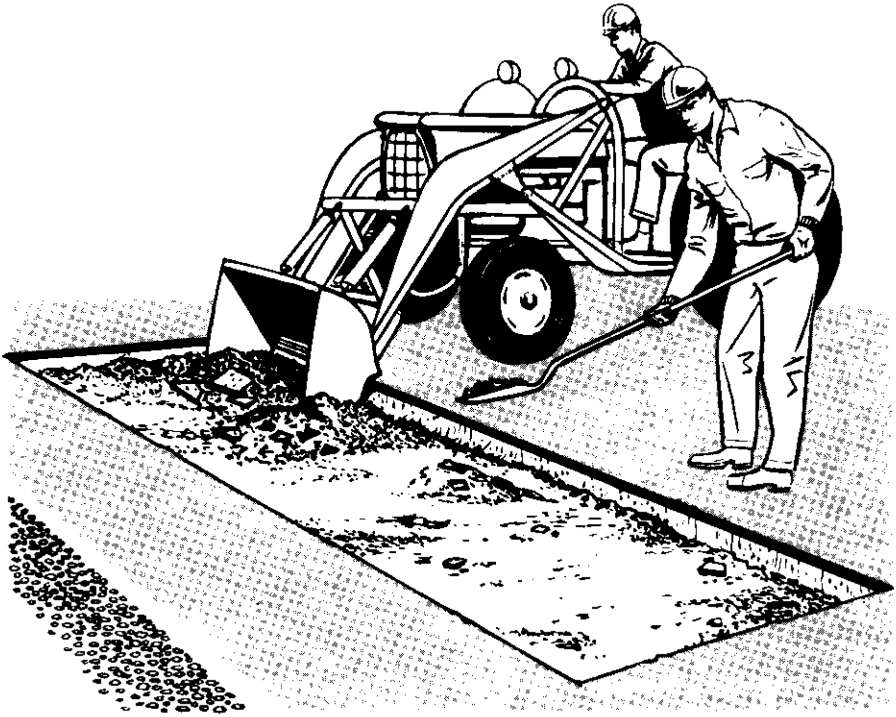

Figure 9.3 Removing the Surface and Base.

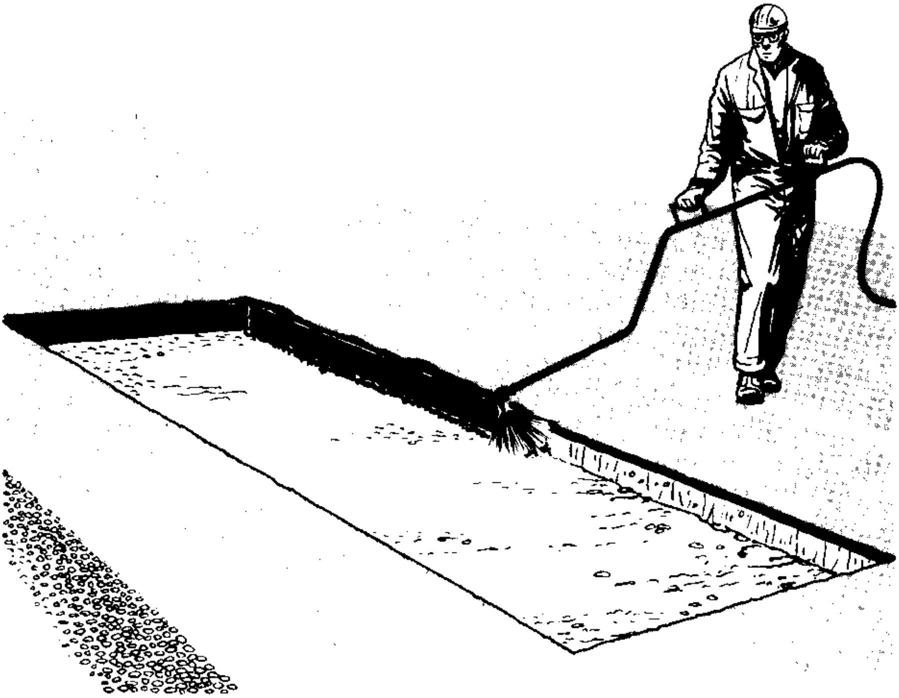

Figure 9.4 Applying a Tack Coat to the Vertical Sides.

For best results, the hole should be backfilled with a dense-graded hot-asphalt plant mix (Figure 9.5), spread carefully to prevent segregation (Figure 9.6). If the asphalt mix is not available, backfill the hole with a good granular base material. Part of the surface and upper portion of the base that was excavated may be broken into small pieces, mixed thoroughly, and placed in the bottom of the hole. If the hole is more than 150mm (6 in.) deep, the backfill should be placed in layers and each layer compacted thoroughly (Figure 9.7). Proper compaction is a major factor in producing a permanent patch. A vibratory-plate compactor is excellent for small patches. A roller may be more practical for large areas. The repair is then completed by placing the hot-mix asphalt surface layer and compacting it flush with the surrounding pavement surface. The patch should not be overfilled in anticipation of traffic compaction. A straightedge or stringline should be used to check the evenness of the surface (Figure 9.8).

Surface Patches

If the area to be repaired consists of small hairline cracks, minor surface distortion or ravelling, a surface or "skin" patch may be the most cost effective technique to employ. The construction of a surface patch does not normally require the removal of existing pavement. Typically, only a layer of hot-mix asphalt or a chip seal cover to the distressed area is required.

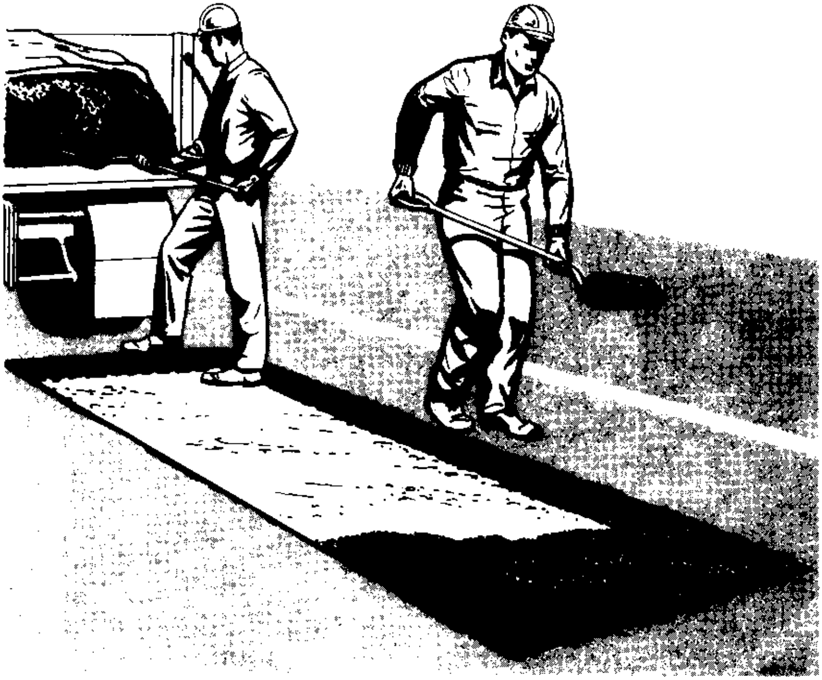

Figure 9.5 Backfilling the Hole With Plant Mix.

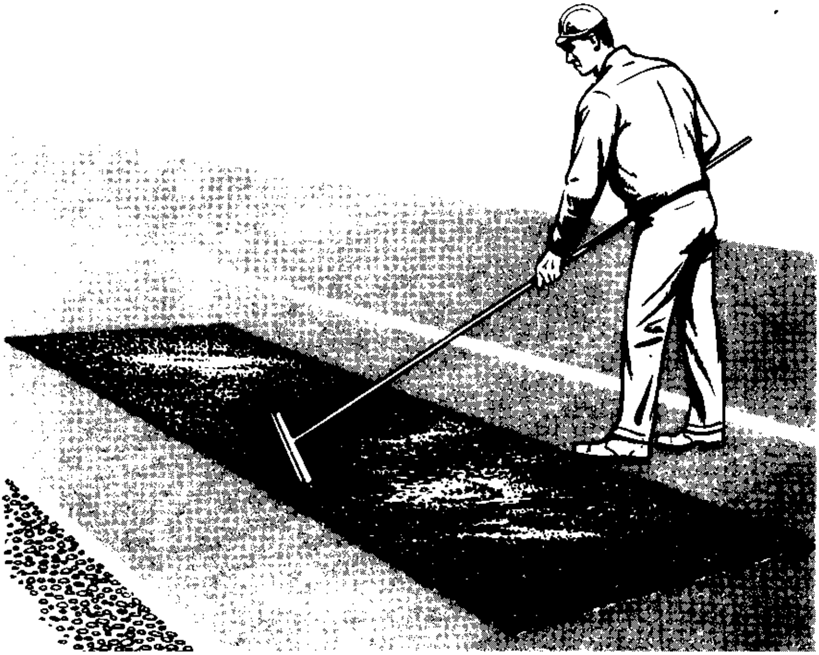

Figure 9.6 Spreading the Mix.

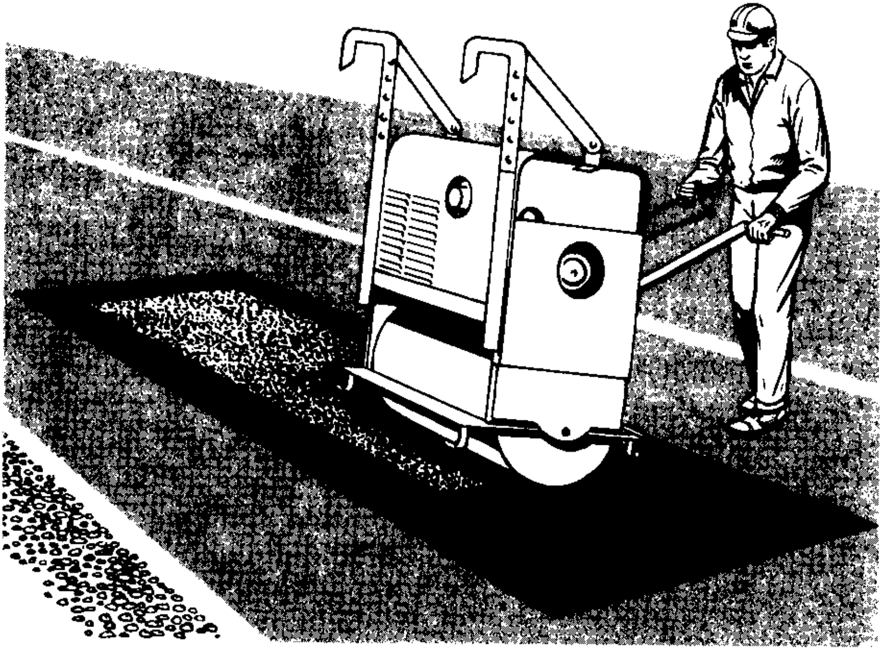

Figure 9.7 Compacting the Mix.

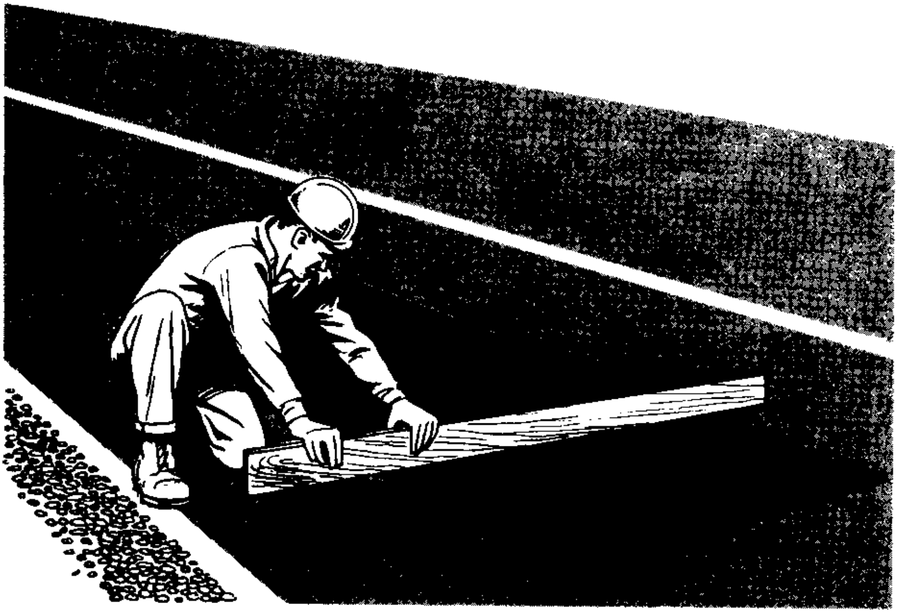

Figure 9.8 Checking the Patched Surface With a Straightedge.

When hot-mix asphalt is used for this purpose, a sand asphalt mixture is preferable. The reason for this is that surface patches are "feather-edged" to a zero thickness and the maximum thickness of a surface patch usually does not exceed 12 to 18 mm (½ to ¾ in.). Thus, the use of mixes containing coarse aggregate should be avoided in this application. The area to be patched should first be cleaned by brooming or blowing with high-pressure air. A tack coat should then be applied over the entire area to be patched. The hot mix is then spread rapidly over the tack coat and immediately compacted with a roller or flat-plate vibratory compactor. The edges should be finished to a neat, clean line.

An alternative procedure for patching with hot mix is to apply a sprayed membrane of emulsified asphalt to the distressed area followed by application of "chips" or aggregate. The aggregate of a uniform size is immediately spread over the emulsion coating. The aggregate should then be embedded with an appropriate compaction device.

Causes and Repairs of Cracks

Crack Sealing

Cracks are sealed for two major reasons:

(1) To prevent the intrusion of incompressible materials; and

(2) To prevent the intrusion of water into the underlying pavement layers.

Many types of satisfactory crack sealing materials are available. However, there are certain general properties that should apply to any material used for this purpose. These properties include:

- Good bonding/adhesion
- Flexibility and Extendability
- Ease of application
- Resistance to softening
- Resistance to tracking
- Resistance to weathering
- Compatibility to asphalt

Crack sealing might be broken into two general classifications.

Temporary crack sealing is done with materials other than modified asphalt or specially prepared crack and joint sealers. These crack sealers (mostly emulsified and cutback asphalts) maintain a tightly sealed crack usually only as long as the pavement remains quite stationary, both longitudinally and vertically. Once expansion and contraction or excessive vertical movement occurs the effectiveness of the seal can be lost and the cracks must be resealed. In climates with severe temperature changes, the resealing may be necessary every year. In moderate climates this will be necessary less often, and in mild climates even less frequently.

Although the cost per unit of this type of crack sealing is relatively low, the frequency with which it must be done might dictate a more lasting solution.

Modified asphalt crack sealing is done with modified asphalts (e.g. rubber-asphalt sealer) or specially prepared crack and joint sealers (also refer to ASTM D 3405). These materials are typically more expensive, requiring highly controlled handling techniques and much more preparation of the crack. However, they do maintain an effective seal for a longer period of time in almost any type of climate. Note, however, that three to eight years is the general expected life, depending on the material used, the condition of the pavement, and the techniques used when applying the crack seal.

Although initially they are more expensive to use, modified asphalt sealers do offer the saving of manpower and time, and might, in the long term, be more cost effective.

Prior to sealing a crack it is absolutely essential that all incompressible material be removed by high pressure air or routing. If grass is growing through the crack it might be necessary to inject an approved liquid herbicide to prevent future growth.

It is usually not practical to individually seal cracks that are less than 6 mm (¼ in.) wide. For small, hairline cracks an asphalt slurry mixture can be squeegeed over the surface and forced into the cracks. The slurry must be maintained at a significant fluidity to be able to flow into these small cracks.

Types of Cracks

Cracking appears in many forms. Simple crack filling may be the right treatment in some cases. In others, complete removal of the affected area and the installation of drainage may be necessary before effective repairs can be carried out. To make proper repairs, causes of the cracking must first be determined.

The cracking pattern is frequently the same for various causes and for various stages of distress. Types of cracks maintenance crews commonly encounter are:

- Alligator cracks
- Edge cracks
- Joint cracks
- Reflection cracks
- Shrinkage cracks
- Slippage cracks.

Alligator Cracks

Alligator cracks are interconnected cracks forming a series of small blocks resembling an alligator's skin or chicken wire (Figure 9.9). They are usually associated with a granular untreated base that has failed or with soft subgrade.

Since alligator cracking is probably caused by a saturated base or subgrade, correction should include removing the wet material and possibly installing drainage. A hot-mix asphalt placed full-depth provides a strong patch. If hot-mix asphalt is unavailable, new granular material compacted in layers, primed and covered with sufficient thickness of hot-mix asphalt may be used.

Skin patches can be used for a temporary repair. In any event, repairs should be made promptly so that further damage to the pavement does not occur.

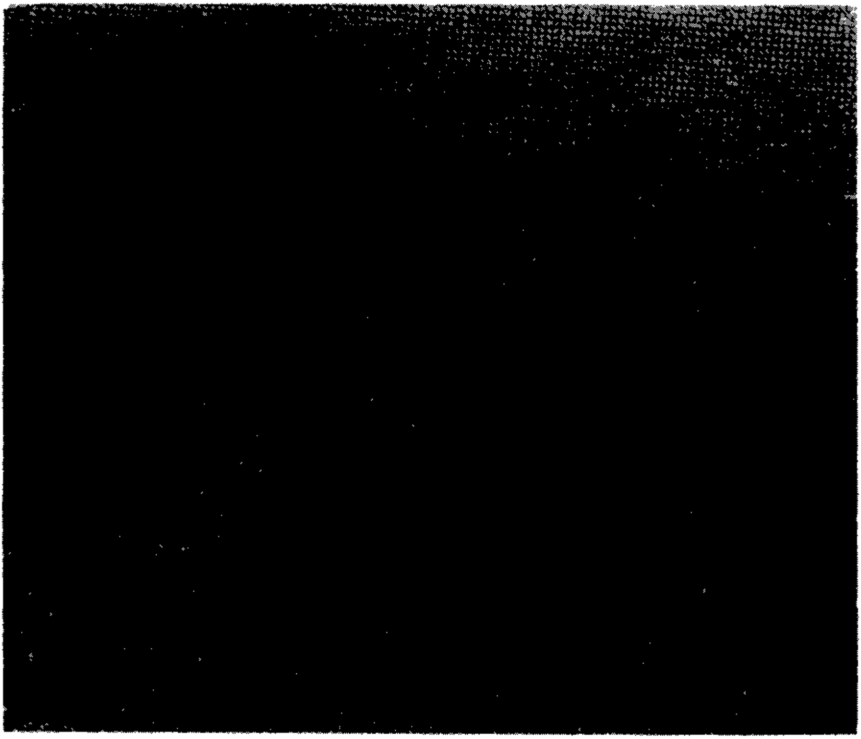

Figure 9.9 Alligator Cracks.

Edge Cracks

Edge cracks are longitudinal cracks near the edge of the pavement, with or without transverse cracks branching towards the shoulder (Figure 9.10). Edge cracks usually are caused by lack of lateral or shoulder support for the asphalt pavement. They may also be caused by settlement or yielding of the base material underlying the cracked area. This in turn may be the result of poor drainage, frost heave, or shrinkage from drying out of the surrounding earth.

For repair, edge cracks may be filled with asphalt emulsion slurry or cutback asphalt mixed with sand. If the edge of the pavement has settled, the surface may be brought to grade with a hot-mix asphalt patching material.

Joint Cracks

There are two types of joint cracks. One is the edge-joint crack, which occurs between the pavement and the shoulder (Figure 9.11). The other is

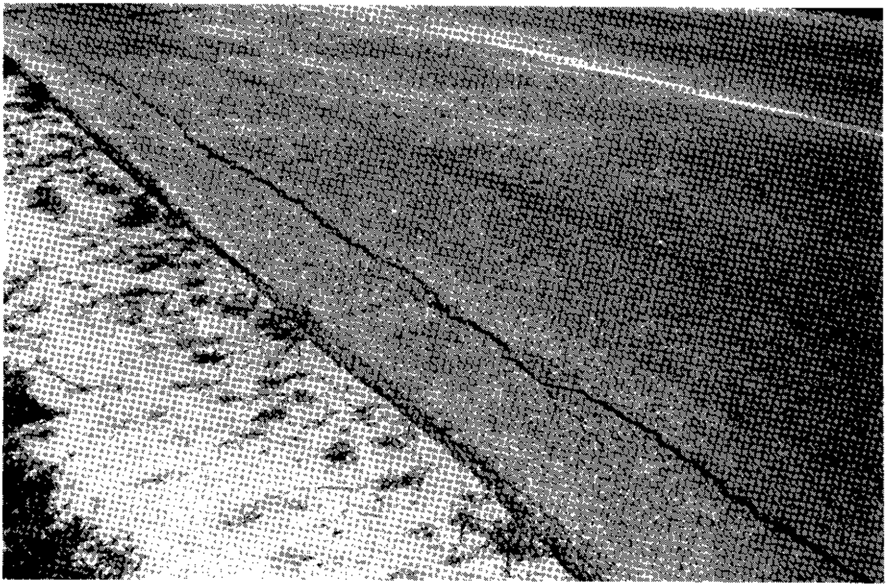

Figure 9.10 Edge Crack.

the lane-joint crack, which occurs between two adjoining paving lanes (Figure 9.12).

Alternate wetting and drying or freezing and thawing beneath the shoulder surface is a common cause of edge-joint cracking. It usually results from poor drainage where water is trapped or ponded in depressions over the pavement-shoulder joint.

Lane-joint cracks, on the other hand, are usually caused by a weak seam or poor bond between adjoining spreads in the pavement.

Where water is a contributing factor to cracking, drainage should first be corrected. The cracks are filled with an asphalt-emulsion slurry. Special asphalt compounds or heavier-bodied asphalts may be used to fill larger cracks.

Reflection Cracks

Reflection cracks (Figure 9.13) occur in asphalt overlays. These cracks reflect the crack pattern in the pavement structure underneath. They are most frequently found in asphalt overlays over portland cement concrete and cement-treated bases.

Reflection cracks are caused by vertical or horizontal movements in the pavement beneath the overlay resulting from traffic loads, temperature, and earth movements.

If the cracks are less than 3 mm (⅛ in.) wide, it may be wise to ignore them unless water can enter and cause further damage. In this case they may be filled by a squeegee technique, using emulsified or cutback asphalt covered with sand. Cracks wider than 6mm (¼ in.) are filled with an asphalt-emulsion slurry or a light grade of cutback asphalt and fine sand.

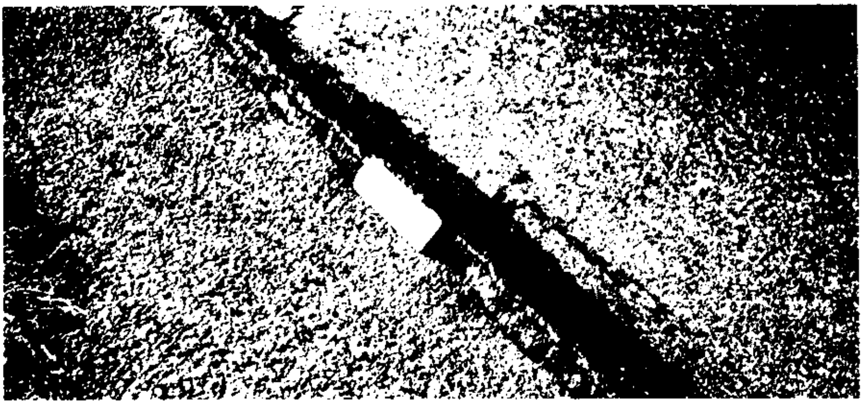

Figure 9.11 Edge Joint Crack.

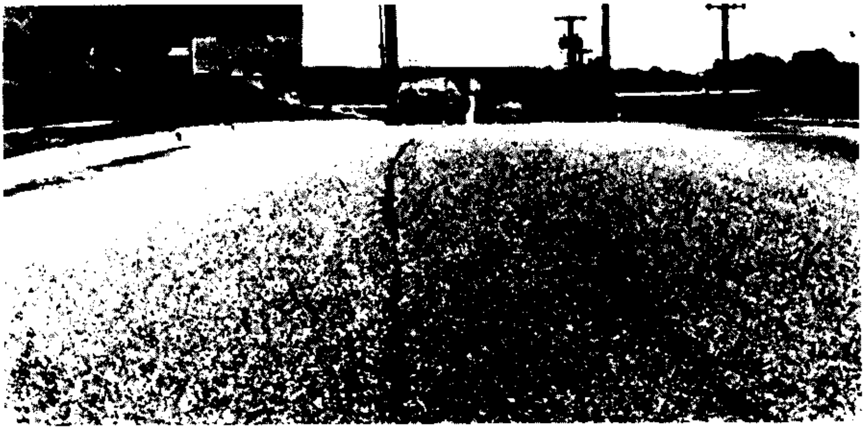

Figure 9.12 Lane Joint Crack.

Shrinkage Cracks

Shrinkage cracks are interconnected cracks forming a series of large blocks, usually with sharp corners or angles (Figure 9.14). Often it is difficult to determine whether shrinkage cracks are caused by volume change in the asphalt mix or in the base or subgrade. Frequently, they are caused by volume change of fine aggregate asphalt mixes that have a high content of low penetration asphalt.

Cracking due to aging is another form of shrinkage cracking. After years of exposure to the elements, asphalt may lose some of its elasticity or resiliency. At the same time the materials in the pavement are subjected to constant expansion and contraction caused by temperature changes. These processes may cause a crack pattern similar to that in Figure 9.14. This type of cracking is not an indication of a structural deficiency.

Shrinkage cracks should be sealed with an emulsion asphalt slurry followed by a surface treatment, a slurry seal, or an overlay on the entire surface.

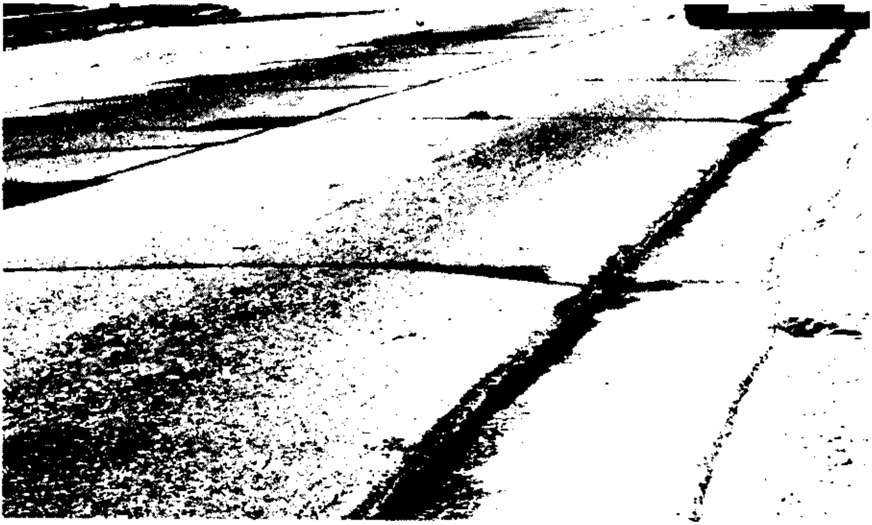

Figure 9.13 Reflection Crack.

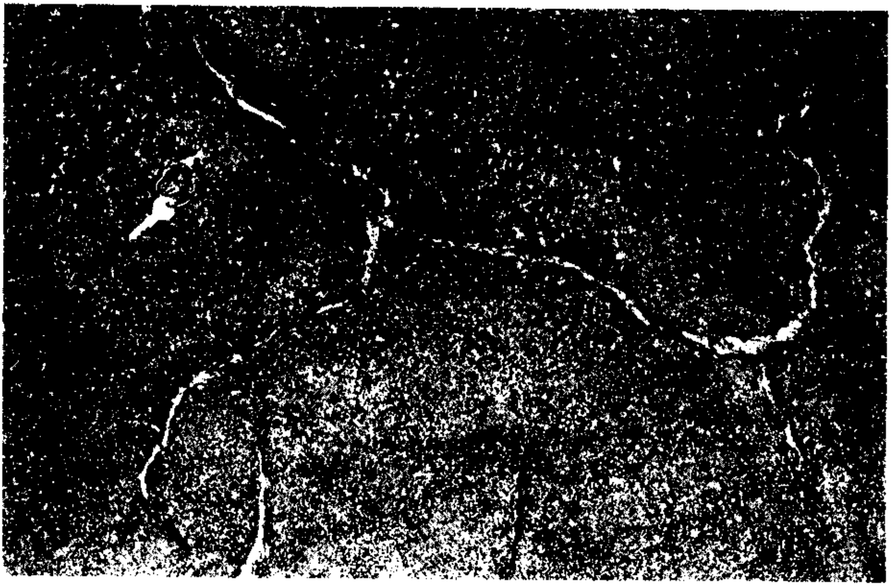

Figure 9.14 Shrinkage Cracks.

Slippage Cracks

Slippage cracks (Figure 9.15) are crescent-shaped cracks resulting from horizontal forces induced by traffic. They are caused by a lack of bond between the surface layer and the course beneath. Lack of bond may be due to dust, dirt, oil, or the absence of a tack coat.

The only effective way to repair a slippage crack is to remove the surface layer from around the crack to the point where there is good bond between the layers. Then the area is patched with a hot-mix asphalt.

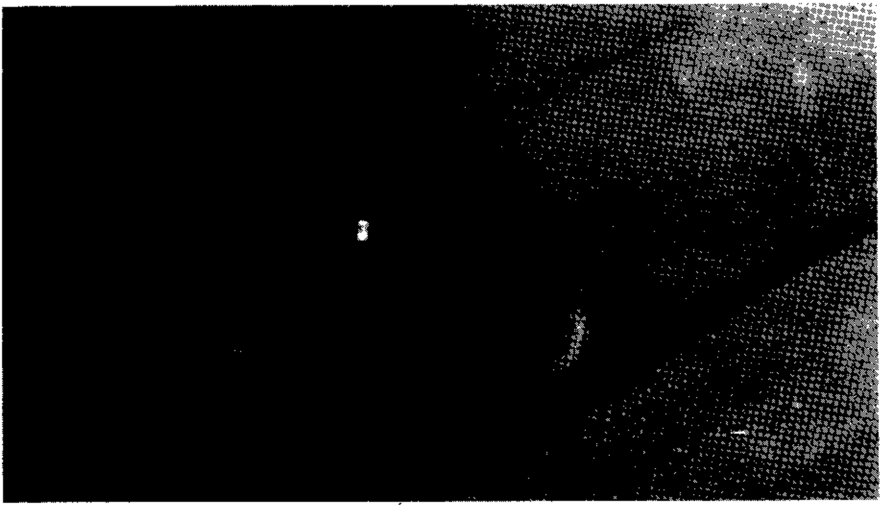

Figure 9.15 Slippage Cracks.

Causes and Repair of Distortions

Pavement distortion is the result of subgrade weakness where additional compaction or movement of the subgrade soil has taken place, or where base compaction has occurred. It may or may not be accompanied by cracking, but in either instance it creates a traffic hazard, permits water to accumulate, and eventually makes matters worse. Distortion takes a number of different forms:

- Channeling
- Corrugations
- Shoving
- Depressions
- Upheaval

As with any other defect, the type of distortion has a cause, and it must be determined before a correct remedy can be applied. Pavement distortion is more likely to occur at intersections and other stop-start areas where traffic, especially heavy trucks, imposes severe horizontal stress on the pavement surface. Repair techniques range from leveling the surface by filling with new material, to complete removal of the affected area and replacing with new material.

Channeling

Channeling, grooving, or rutting are channelized depressions that develop in the wheel tracks of asphalt pavements (Figure 9.16). Channeling may result from consolidation or lateral movement under traffic in one or more of the underlying courses, or by displacement in the asphalt surface layer itself. It may develop under traffic in new asphalt pavements that had too little compaction during construction, or from plastic movement in a mix that does not have enough stability to support traffic.

The cause for the channeling should be determined. In some cases it may be necessary to remove the pavement layer identified as the cause. In less severe situations the ruts or channels might be milled to a level plane or simply filled with hot-mix asphalt to be followed by an overlay across the entire pavement. Failure to determine and correct the cause may result in inadequate, short-lived maintenance.

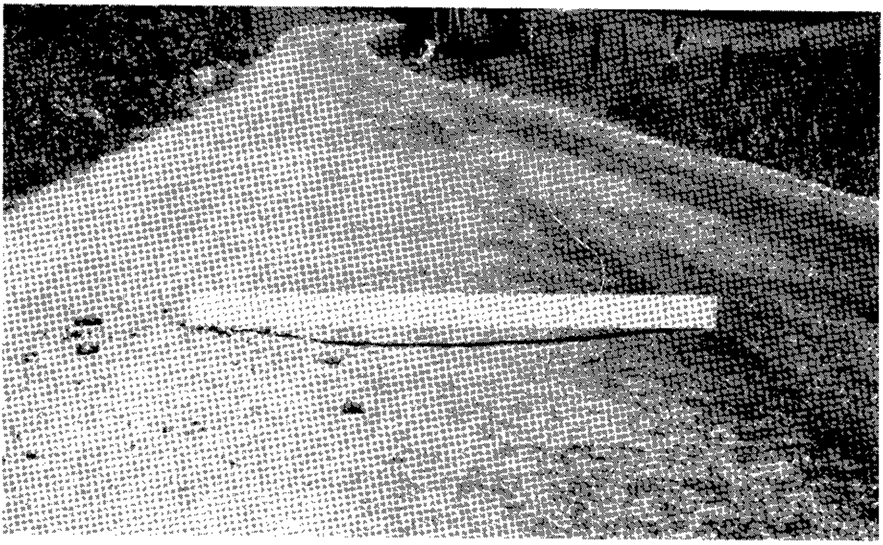

Figure 9.16 Channeling.

Corrugations and Shoving

Corrugations, or washboarding, is a form of plastic movement typified by ripples across the asphalt pavement surface (Figure 9.17). Shoving (Figure 9.18) is plastic movement resulting in localized bulging of the pavement surface. Both corrugations and shoving usually occur at points where traffic starts and stops, or on hills where vehicles brake on the downgrade.

Corrugations and shoving also usually occur in asphalt pavement mixtures that lack stability. This may be the result of too much asphalt, too much fine aggregate, improper gradation, or round or smooth textured coarse aggregate. In the case of emulsified and cutback asphalt mixes, it may be due to a lack of aeration.

If the corrugated pavement has an aggregate base with a thin surface treatment, the surface should be scarified, mixed with the base, and recompacted before resurfacing. If the surface is more than 50mm (2 in.) thick, shallow corrugations can be removed with a cold milling machine (Figure 9.19). The area is then given a seal coat or hot-mix surface.

For effective repair, shoved areas should be removed and patched.

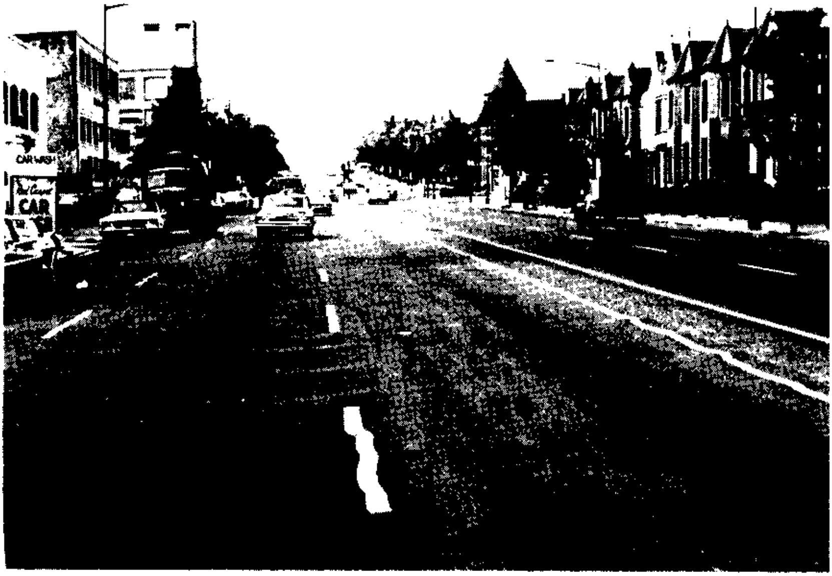

Figure 9.17 Corrugations.

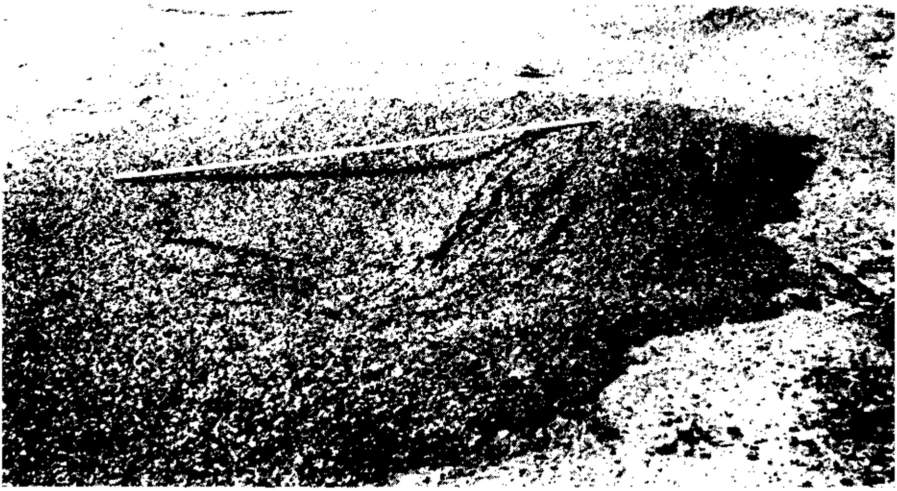

Figure 9.18 Shoving.

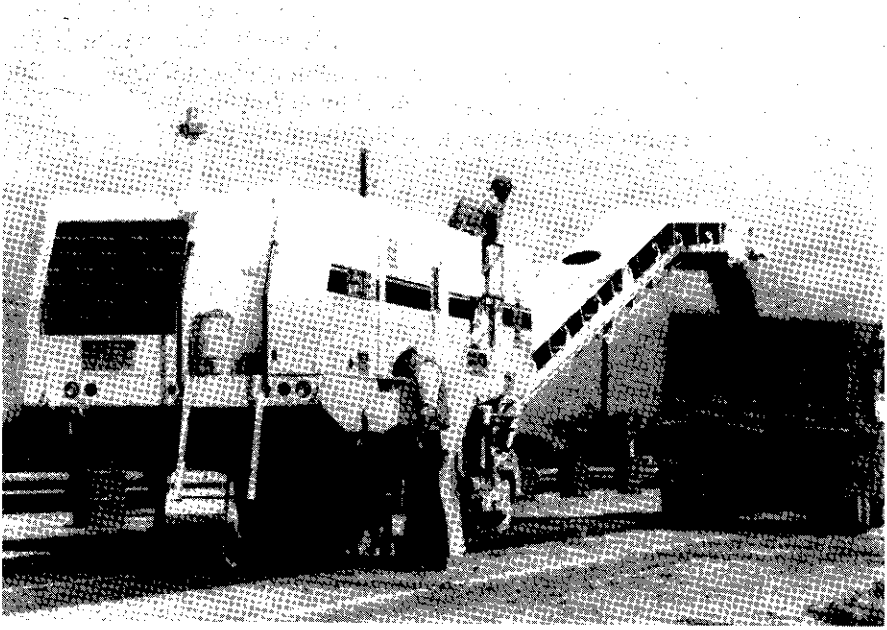

Figure 9.19 Cold Milling Machine. *(Courtesy CMI Corporation)*

Depressions

Depressions are localized areas of limited size that may or may not be accompanied by cracking (Figure 9.20). Water collects in depressions, which then become not only a source of pavement deterioration but a hazard to motorists. Depressions are caused by traffic heavier than that for which the pavement was designed and by consolidation or movement within the subgrade.

Depressions should be filled with hot-mix asphalt and compacted to restore the area to the same grade as the surrounding pavement.

Upheaval

Upheaval is the localized upward displacement of a pavement due to swelling of the subgrade or some portion of the pavement structure (Figure 9.21). It is most commonly caused by ice expansion in the granular courses beneath the pavement or in the subgrade. Upheaval may also be caused by the swelling effect of moisture on expansive soils.

Correction of ice expansion areas requires that the water source be isolated from freezing temperatures or that the water be drained from beneath the pavement. The water source can be isolated from freezing temperatures by placing layers of insulation under the pavement structure or by replacing the native subgrade material with free draining granular material to a depth below the frost line. If the water source is localized, an

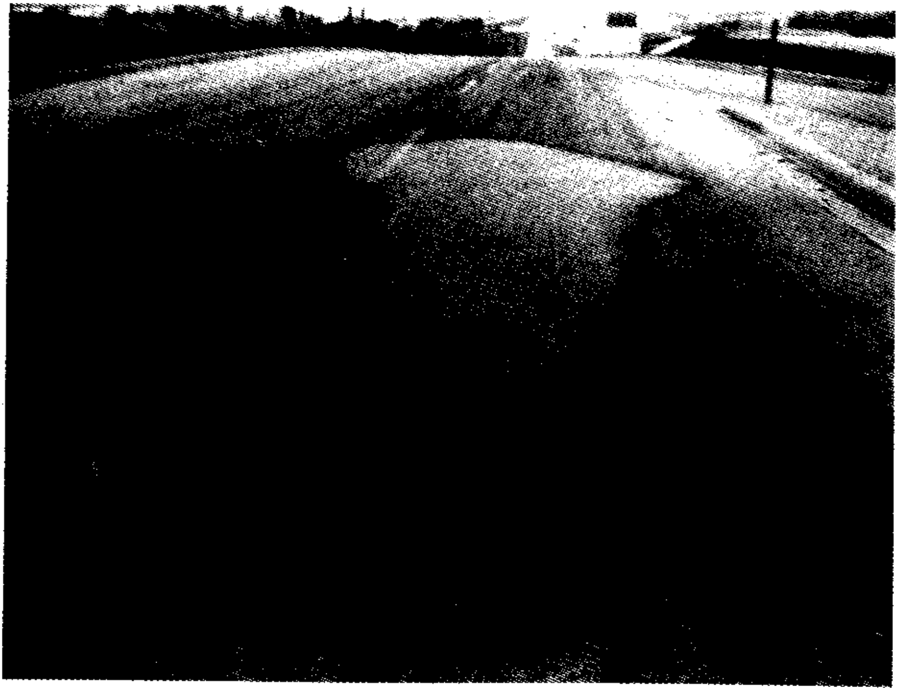

Figure 9.20 Depression. The Depressed Area is Denoted by Water Lying on the Pavement.

under drain system may be effective. In either case the pavement must be removed and replaced to allow permanent repair.

Upheaval caused by expansive soils may be arrested by installing drainage systems to remove water access to the subgrade. Remedial maintenance to restore ride quality can be done by cold milling the surface and placing a surface treatment or a thin overlay. If drainage is not successful, complete pavement removal and reconstruction may be the only solution.

Causes and Repair of Disintegration

Disintegration is the breaking up of a pavement into small, loose fragments. This includes the dislodging of aggregate particles. If not stopped in its early stages, disintegration can progress until the pavement requires complete rebuilding. Pot holes and raveling are two of the more common types of early stage disintegration. Repair ranges from simple seals in early stages to deep patches if allowed to progress to a severe level.

Pot Holes

Pot holes are bowl-shaped holes (Figure 9.22) of various sizes in the pavement resulting from localized disintegration under traffic. They are

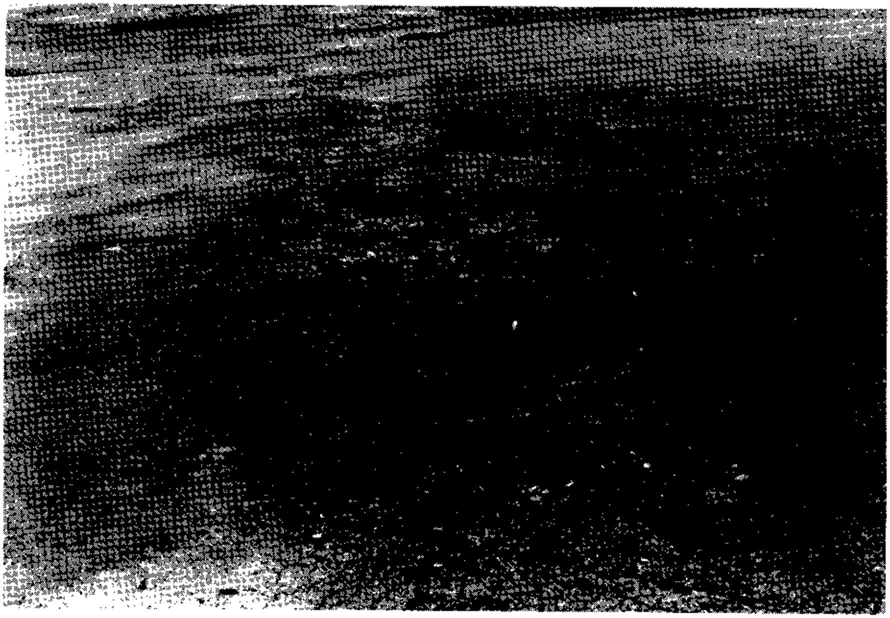

Figure 9.21 Upheaval.

usually caused by weakness in the pavement resulting from a weak subgrade or base course, too little asphalt, too thin an asphalt surface, too many fines, too few fines, or poor drainage.

Pot holes frequently appear when appropriate maintenance is not promptly applied once the distressed conditions have been detected. Temporary repair usually involves cleaning out the hole and filling it with a premixed asphalt patching material. Permanent repair is made by constructing a deep patch with asphalt hot-mix.

Raveling

Raveling is the progressive loss of surface material by weathering and/or traffic abrasion (Figure 9.23). Usually the fine aggregate wears away first, leaving little pock marks in the pavement surface. As erosion continues, larger particles eventually break free and the pavement soon has the rough and jagged appearance typical of surface erosion. Raveling is caused by poor construction methods, inferior aggregates, or poor mix design. An early application of a fog seal when raveling is first detected will generally retard progressive deterioration.

Dry and weathered surfaces usually require a surface treatment. Emergency measures include a fog seal applied over the roadway surface. Surface treatments include slurry seal, sand seal, or asphalt-aggregate treatment, depending on the surface condition and traffic.

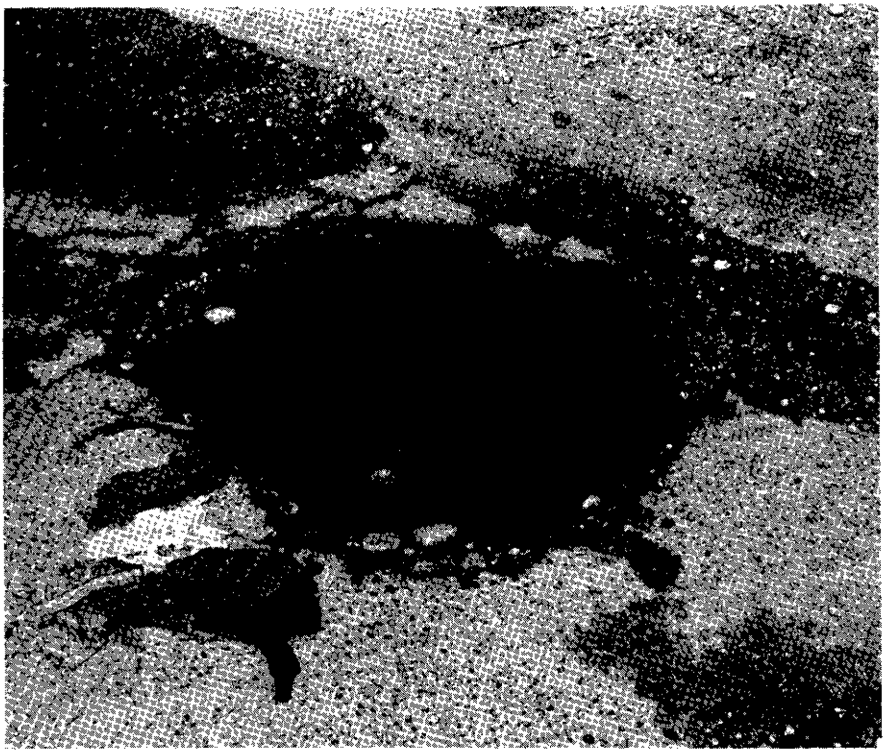

Figure 9.22 Pot Hole.

Causes and Repair of Slippery Surfaces

Excess Asphalt

Bleeding or flushing is the presence of excess asphalt or a film of asphalt on the pavement surface (Figure 9.24). Pavement courses having rich asphalt mixes, improperly constructed seal coats, or too heavy a prime or tack coat may cause bleeding and flushing. Heavy traffic loads can force or flush the asphalt to the surface of the pavement in hot weather. If hydrophilic aggregates are used in the hot-mix asphalt, unfavorable moisture conditions may cause stripping with subsequent migration of the asphalt to the pavement surface.

In many cases, bleeding can be corrected by repeated applications of hot sand, rock screenings or coarse aggregate to blot up the excess asphalt. When this technique is employed the aggregate used should be at least twice the diameter of the asphalt film thickness. It is more effective if the aggregate can be heated to approximately 230° C (450° F) prior to being applied to the bleeding area. After the aggregate is applied it should be rolled into the

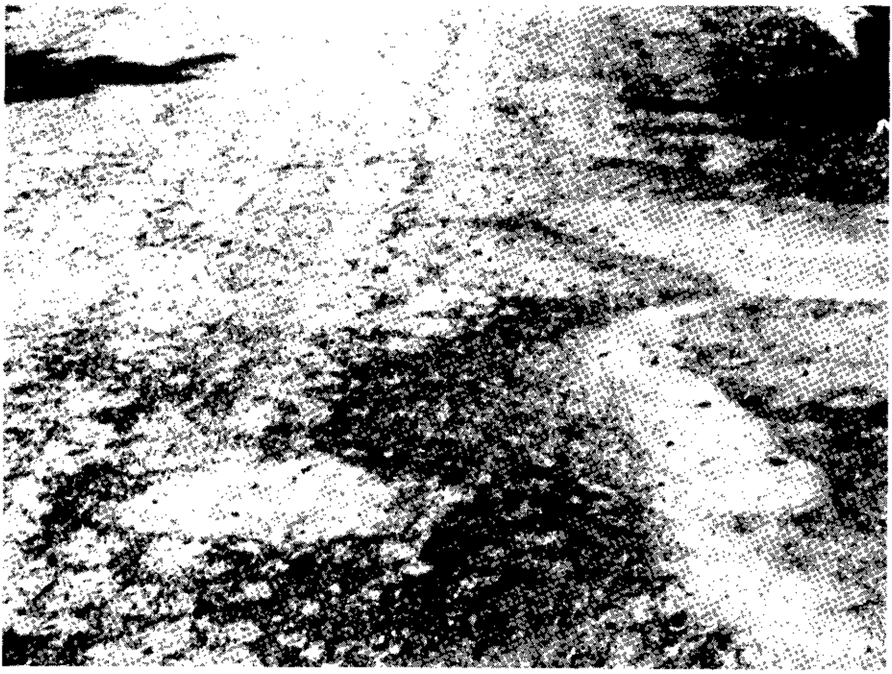

Figure 9.23 Raveling.

asphalt film. If the bleeding is light, a hot-mix asphalt friction course or an aggregate seal coat using absorptive aggregate might be used.

A pavement planing machine, either cold or hot, may be used to remove excess asphalt or to remove the entire layer.

Polished Aggregate

Aggregate particles in the surface of a pavement may be worn smooth under the abrasive action of traffic (Figure 9.25). This includes both naturally smooth uncrushed gravels and crushed rock that wears quickly under the action of traffic. Some aggregates, particularly some types of limestone, become polished rather quickly under traffic. Others, such as some types of gravel, are naturally polished, and if they are used in a pavement surface without crushing they become a skid hazard. Polished aggregates are quite slippery when wet.

The only effective way to repair a pavement with polished aggregate is to cover the surface with a skid-resistant treatment. A hot-asphalt plant-mixed thin overlay friction course, a sand seal, or an aggregate seal should be applied. The aggregate must be hard and angular, such as slag, silica sand, or other proven nonpolishing materials.

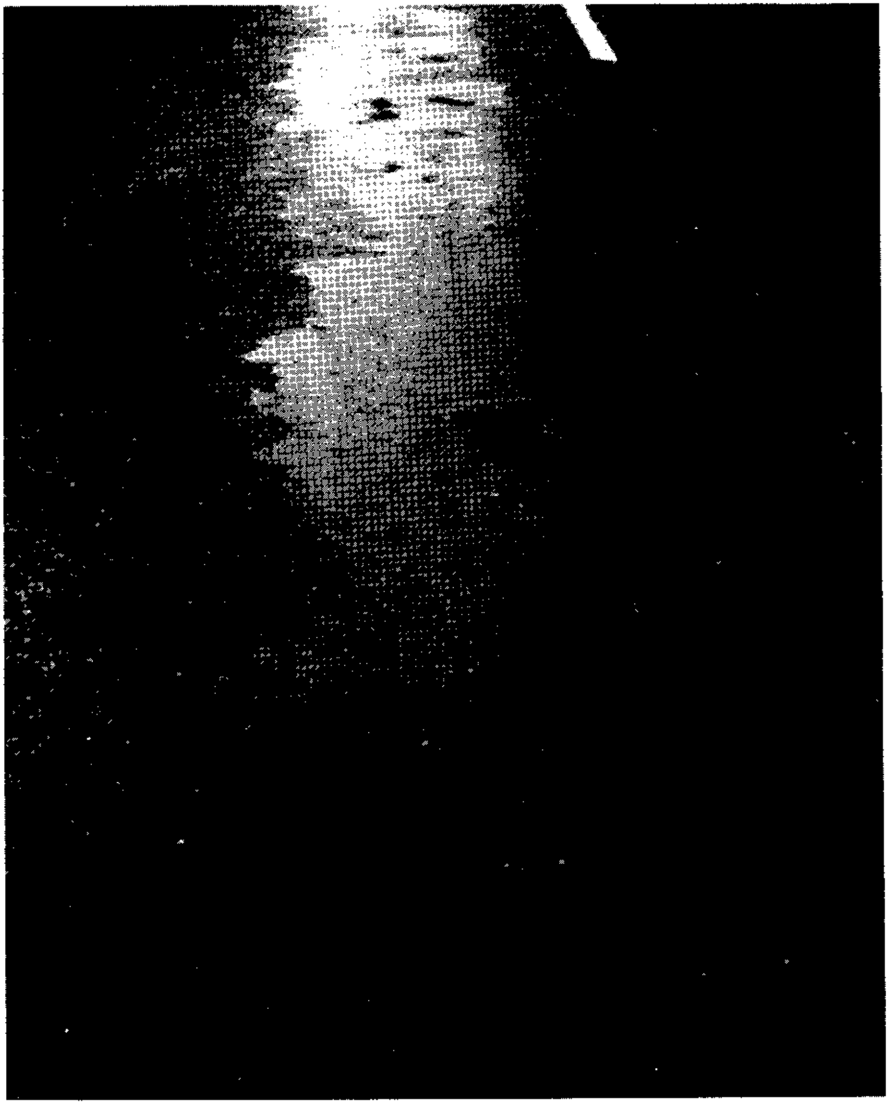

Figure 9.24 Bleeding Asphalt.

Surface Treatment Problems

Because of construction methods, surface treatments may develop some defects that do not occur in other types of asphalt pavement surfaces. These include loss of cover aggregate and streaking.

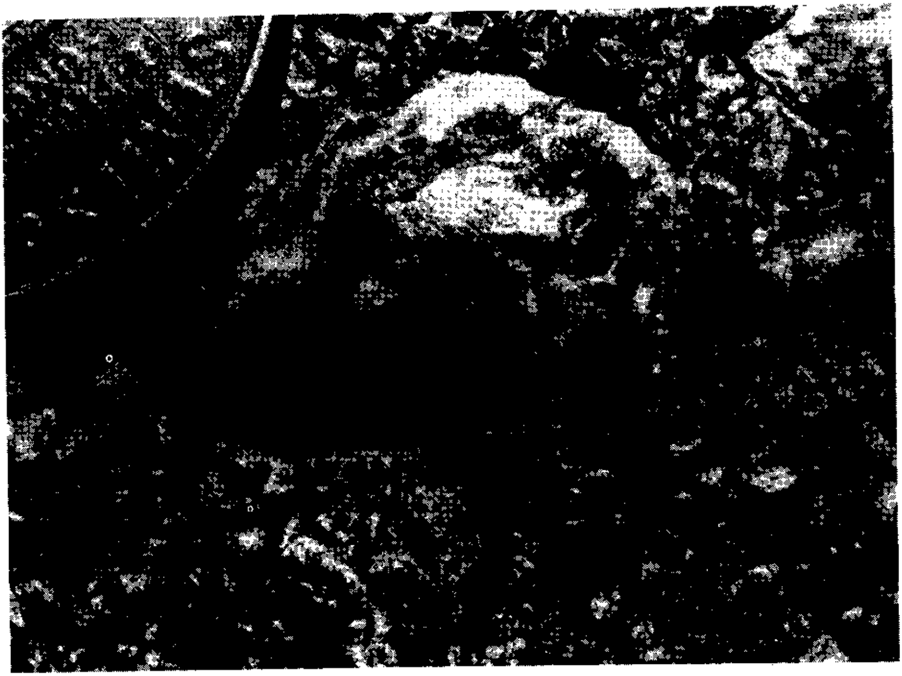

Figure 9.25 Polished Aggregate in Pavement Surface.

Loss of Cover Aggregate

Loss of cover aggregate is the whipping-off of aggregate under traffic from a surface-treated pavement, leaving the asphalt film exposed (Figure 9.26). Several things can cause the loss of cover aggregate:

- Aggregate spread after asphalt has cooled too much
- Aggregate too dusty or wet when spread
- Aggregate not rolled or seated immediately after spreading
- Steel-wheel roller bridged over low spots
- Traffic allowed on new surface treatment too soon
- Not enough or wrong grade of asphalt or an absorbent surface
- Over-rolling, particularly with a steel-wheeled roller.

Hot, coarse sand spread over the affected area may be tried to replace lost aggregate. It should be rolled immediately with a pneumatic roller so that it is seated in the asphalt.

Longitudinal Streaking

Longitudinal streaking (Figure 9.27) is alternating lean and heavy lines of asphalt running parallel to the center line of the road. At the time of asphalt spraying, the spray pattern did not provide a uniform coverage, hence the longitudinal streaking. Several things can cause this, such as:

- Spray bar not set at proper height above the ground

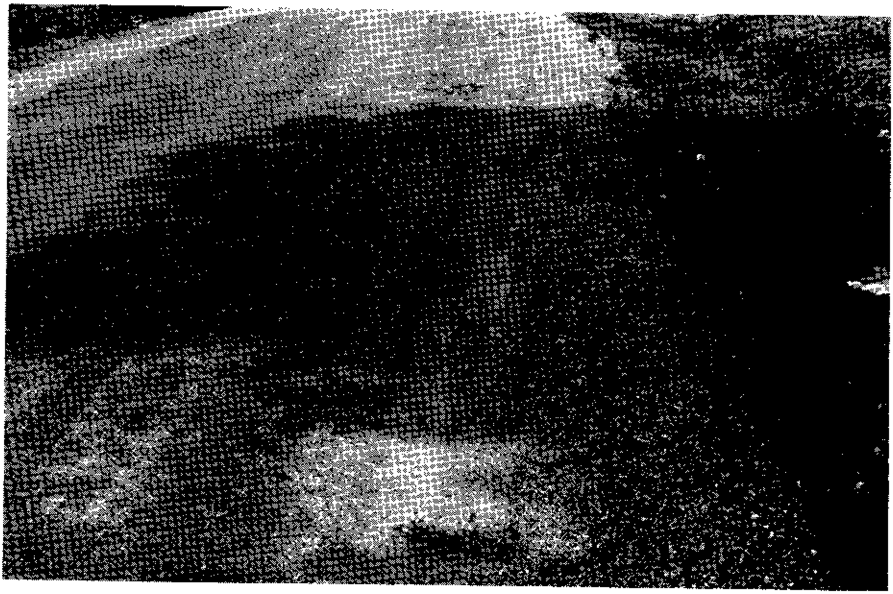

Figure 9.26 Loss of Cover Aggregate.

- Nozzles on spray bar not set at correct angle or are the wrong size, plugged, or defective
- Wrong asphalt pump speed
- Asphalt too cold or too high in viscosity
- Too low pump pressure.

About the only satisfactory repair for longitudinal streaking is to plane off the streaked surface and apply a new surface treatment. It is much easier to prevent longitudinal streaking than to correct it. Careful adherence to the manufacturer's recommendations for the asphalt distributor before and during use will forestall streaking. Good inspection practices during the construction process will also minimize the potential for streaking.

Transverse Streaking

Transverse streaking is alternating lean and heavy lines of asphalt running across the road that may result in corrugations in the pavement surface. It is caused by spurts in the asphalt spray from the distributor spray bar. These spurts may be produced by pulsation of the asphalt pump due to worn or loose parts, by improper pump speed, or by a miss in the motor.

Again, about the only satisfactory repair is to plane off the streaked surface and apply a new surface treatment.

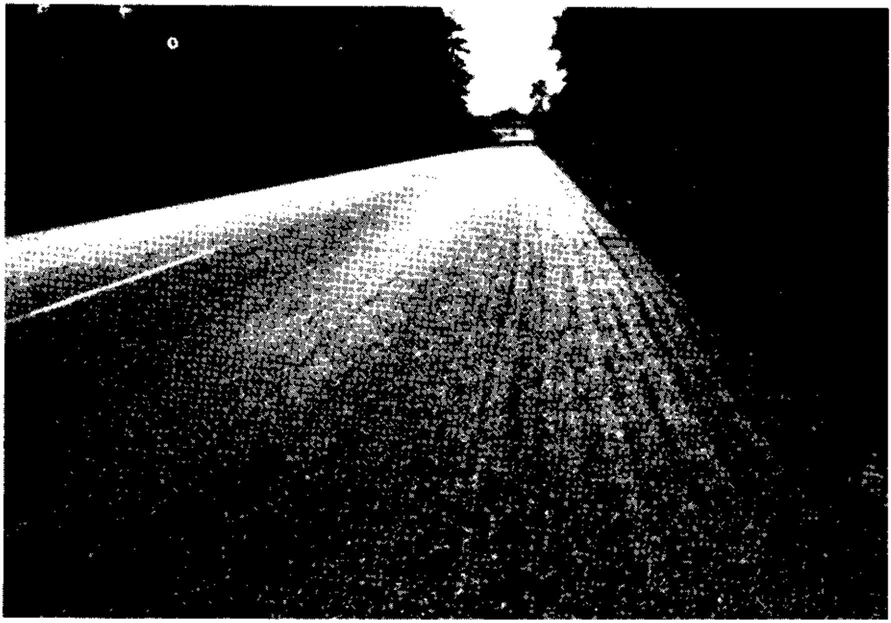

Figure 9.27 Longitudinal Streaking.

9.3 Asphalt in Rigid Pavement Maintenance

Introduction

Asphalt for PCC Maintenance

Asphalt plays an important role in the maintenance of portland cement concrete (pcc) pavements. It seals joints and cracks, fills spall cavities, and raises sunken slabs. It also renews disintegrating pavements and covers slippery surfaces. The use of asphalt overlays, crack relief layers and similar procedures are discussed in the rehabilitation section. Asphalt, if used correctly, does an excellent job in repairing many defects in portland cement concrete. The techniques discussed here are not the only correct ones, but they are methods that have proven satisfactory.

The details of patching mixtures, tack coats, the placement of patching mixtures, and the compaction of patches, as discussed earlier, are equally applicable in the patching and repair of portland cement concrete pavements.

Joint and Crack Sealing

Joint and crack sealing is done for several reasons. It prevents surface water seepage, it protects joint fillers, and it keeps out foreign matter. At one time, asphalt cement with or without mineral filler was the most commonly used sealing material. Rubber-asphalt compounds have gained favor because they have less tendency to become brittle in winter or to soften under traffic in summer.

Equipment used for sealing joints and cracks varies, however, truck- or trailer-mounted pressure applicators with hand wands are preferred.

Before any sealer is used, joints and cracks must be cleaned out. Air compressors, sand blasters, and power routers make this job much easier. Just enough sealing material is placed in the joint or crack to fill it. Hot sealer may shrink somewhat on cooling, so enough additional sealer should be added to fill the opening flush with the surface. If the pavement is to receive traffic immediately after sealing, the material must be protected against pickup by tires. This is done by dusting the filled joints and cracks with fine sand, mineral filler, or like material. Cold applied sealers may be covered with paper tape.

Undersealing

Subgrade cavities may be difficult to detect in their early stages. One way to find them is to look for slab movement under loads as soon as the surface has dried following a heavy rain. When cavities exist, slab movement is indicated by water seeping or pumping up through the joints or along the edges of the slab. Other symptoms include a hollow rumbling as a vehicle passes over a cavity, and the rough-riding joint of a settled slab.

Early undersealing of these slabs is highly desirable as preventive maintenance. Properly timed, undersealing stops pumping and distortion of the pavement surface. Undersealing at this time usually postpones the need for resurfacing for several years. But if the surface becomes rough before corrective action is taken, it is advisable to follow undersealing with an asphalt-concrete resurfacing. In all cases, it is well to examine the pavement when it becomes necessary to resurface to see if additional undersealing is necessary.

Asphalt cement for undersealing portland cement concrete pavements has a high softening point, 82-93° C (180 to 200° F), and other properties meeting ASTM D 3141 requirements. It is generally heated from 205-230° C (400 to 450° F) before pumping operations begin. During undersealing, pumping pressures 172 to 414 kPa (25 to 60 psi) may be reached.

Holes having a diameter of 38 to 50 mm (1½ to 2 in.) are drilled through the concrete slab. They should be spaced about 3 m (10 ft) apart along each traffic lane and 1 m (3 ft) from the center line of the pavement. Additional holes are drilled 1 m (3 ft) on each side of all transverse cracks or joints. If corner pumping is excessive, holes are drilled 1 m (3 ft) on each side of the joint or crack and about 1 m (3 ft) from the edge of the pavement. During the pumping procedure, care must be exercised not to "over seal." Some agencies monitor the relative height of one slab to another and immediately stop the pumping of asphalt when the slab starts to rise in relation to its proper elevation.

Extra precautions should be taken in handling the extremely hot asphalt hose and nozzle. Safety masks and protective gloves as well as heavy clothing must be worn by the workmen to protect them from hot asphalt. No greater pressure than necessary should be used in pumping the asphalt. Particular care must be exercised when the nozzle is withdrawn from the hole, as passive or residual pressure may cause asphalt to be blown from the nozzle.

Joint and Crack Maintenance

Relatively large volume changes are produced in pcc by temperature variations. Because of this, all such pavements crack. Cracking is a natural property of the material. Rigid pavements have joints at frequent intervals, enabling the concrete to crack in neat, straight lines.

Joints either extend the full depth of the slab or they are grooved to a depth of ¹/₆ to ¼ of the thickness. Full-depth joints are junctions between slabs to permit expansion or the meet construction requirements. Grooved joints are formed or cut to induce cracking along the weakened plane. They may run longitudinally, transversely, or diagonally in the roadway.

Another joint is found between the concrete slab and the shoulder. It is longitudinal and called a shoulder joint. There are other types of joints having special purposes. They are maintained in the same way as the main types.

The methods used for sealing both joints and cracks are essentially the same. These consist of cleaning out the upper portion of the crevices and filling them with a sealing material. The sealant must stick to both sides of the opening to be effective. Sand, gravel, dust, and trash that may have accumulated in the opening should be removed. Otherwise, when the joint closes under expansion, it may cause the slab edges to spall. If the crack is very narrow or has badly spalled edges, a recess to receive the sealant may be grooved with a pavement saw or rotary cutter.

Resealing Joints

Joints need periodic maintenance. The joint sealant may need replacing for any number of reasons. For example, the wrong type of material may have been used; the sealant may have been overheated; or the joint may not have been thoroughly cleaned before sealing.

In all cases, joints that need resealing should be plowed out to a depth of 25 mm (1 in.). Before the new sealant is applied, all surfaces in and around the joints should be clean and dry, with no bits of filler or old seal clinging to the sides. Sponge rubber, plastic, or tape can be inserted into the groove to provide a nonadhesive lower face for the sealant.

Often, sand and pebbles become lodged in the joint sealing material, and when the slabs expand the seal pushes up from the joint and forms a bump. Sometimes there is enough foreign matter in the joint to cause spalling of the slabs. When this happens, the sealant should be plowed out and the joint resealed. However, in many cases, the excess can be removed by cutting it off with a heated square-point spade or a power-driven rotary cutter.

Sealing Cracks

Cracks in pcc pavements are separation in the pavement due to natural causes or traffic action. Included under this definition are:

- Transverse cracks
- Longitudinal cracks
- Diagonal cracks
- Corner cracks
- Restraint cracks

If a crack is not wide enough to receive sealing material with ease, sealing should not even be tried. Wide cracks should be thoroughly cleaned before sealing.

Transverse Cracks

Transverse cracks (Figure 9.28) appear at approximately right angles to the center line of the pavement. Some major causes of transverse cracks are overloads, repeated bending of pumping slabs, failure of soft foundations, frozen joints, lack of joints, too shallow joints, and shrinkage of the concrete.

The cracks should be cleaned of all loose matter by routing, compressed air, or sandblasting the vertical faces of the crack to a depth of at least 25 mm (1 in.). After cleaning out the crack the loose material should be removed with compressed air. The crack can then be filled with a rubber-asphalt sealer. If the crack is caused by pumping, the area should be given an asphalt underseal.

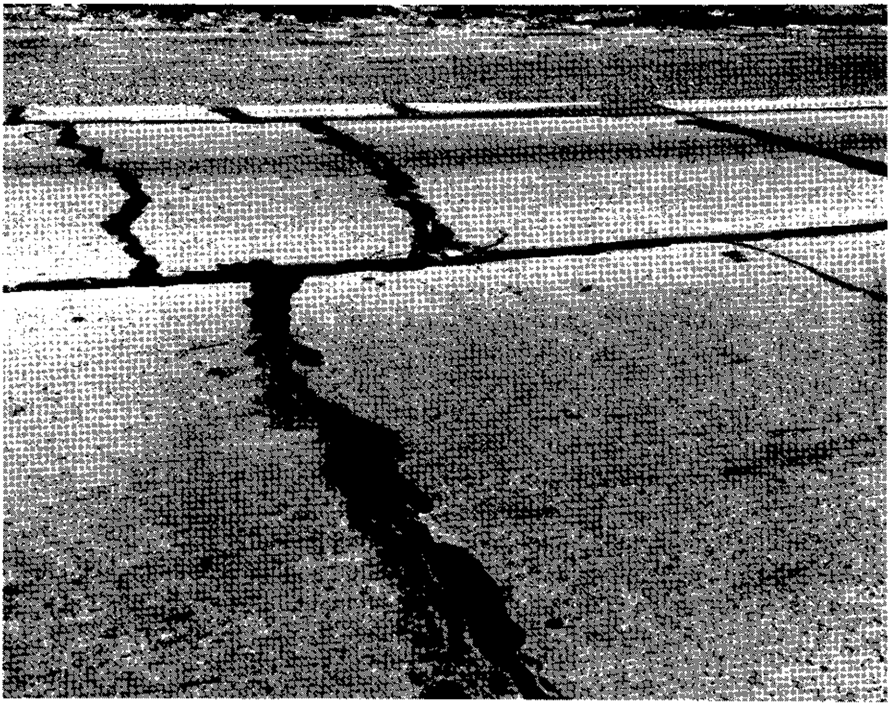

Figure 9.28 Transverse Cracks.

Longitudinal Cracks

Longitudinal cracks run approximately parallel to the center line of the pavement (Figure 9.29). Some causes of this type of cracking are shrinkage of the concrete (if the pavement has no longitudinal joint and is wide), expansive subbase or subgrade, warping stresses in combination with loads, too shallow center-line joints, and loss of support from edge pumping.

To repair the procedure is the same as for transverse cracks; namely, cleaning the crack and filling it with a rubber-asphalt sealer. If pumping is a contributing cause, the void beneath the pavement slab must be filled with a high-softening-point asphalt.

Diagonal Cracks

Diagonal cracks run diagonal to the center line (Figure 9.30). They are caused by traffic loads on unsupported slab ends. The foundation settles or the slab curls, then the subgrade soil pumps out, mostly along the edge, and a diagonal crack results.

The vertical faces of the crack are sandblasted and cleaned out with

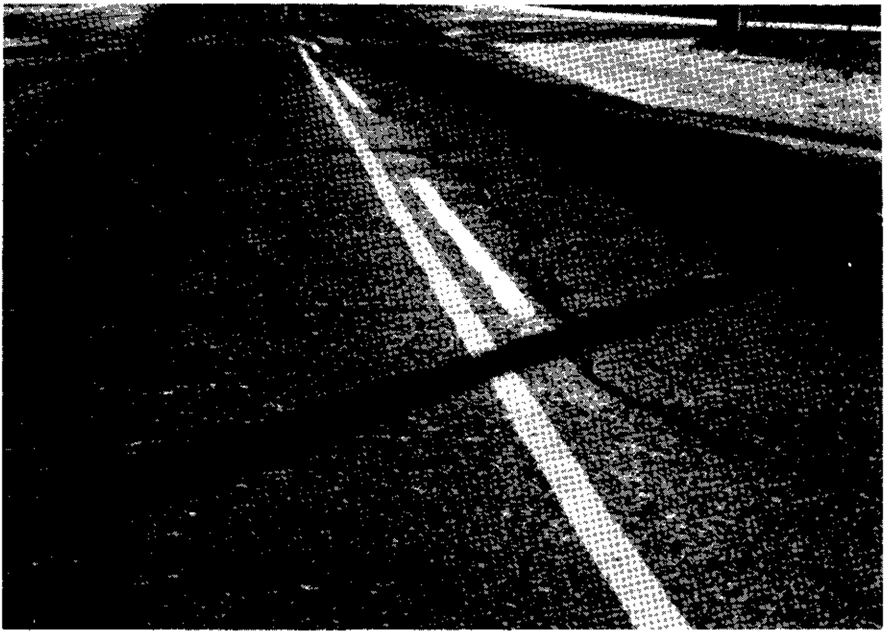

Figure 9.29 Longitudinal Cracks.

compressed air. The crack is then filled with a rubber-asphalt sealing compound. An asphalt underseal is applied to the slab, after which the rubber-asphalt sealing compound is used to finish filling the crack.

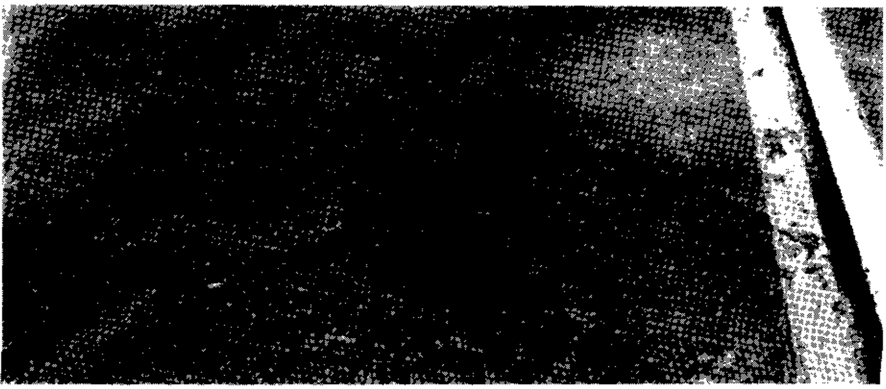

Figure 9.30 Diagonal Cracks.

Corner cracks are diagonal cracks forming a triangle with a longitudinal edge joint and a transverse joint or crack (Figure 9.31). They can be caused by traffic loads on unsupported corners or curled or warped slabs. They may also be caused by loads over weak spots in the subgrade under the slabs.

Corner cracks are best repaired by removing the broken concrete material in the corner and patching with a dense-graded hot-mix asphalt.

Restraint cracks (Figure 9.32) develop within 1 m (3 ft) or less of the outside edges of the pcc pavement and progress in an irregular path toward the longitudinal joint. They are caused by foreign matter such as hard gravel becoming lodged deep in a transverse joint and preventing the slabs from expanding.

The blocked transverse joint should be routed out and resealed. Restraint cracks should be cleaned and sealed with a rubber-asphalt compound. If the cracks are not wide enough to require sealing, they are usually left alone.

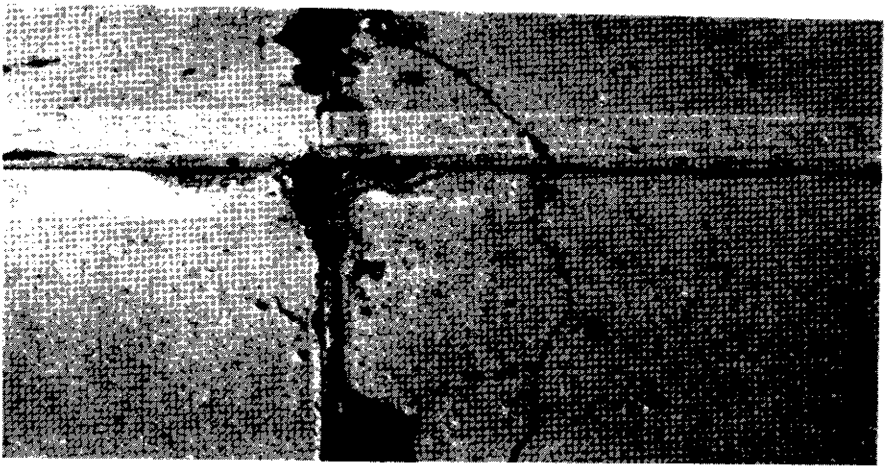

Figure 9.31 Corner Crack and Spall.

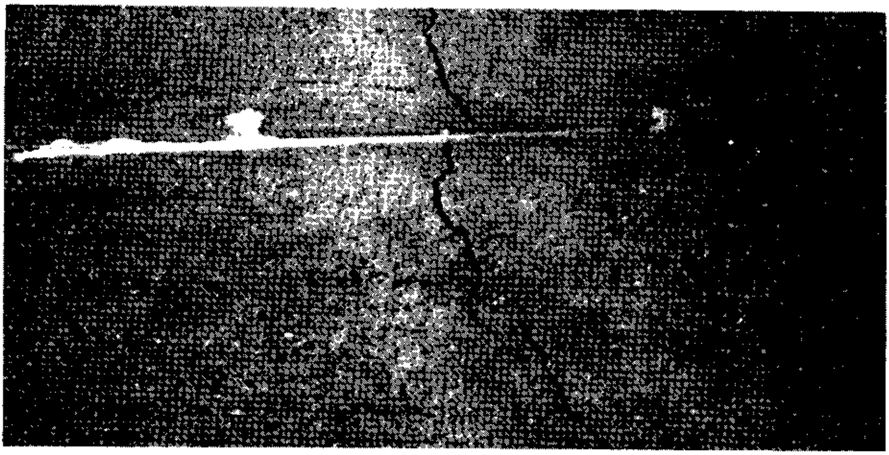

Figure 9.32 Restraint Crack.

Causes and Repair of Distortion

Distortion

Distortion is any change of the pavement surface from its original shape. Faulting is one of the most predominant types of portland cement concrete pavement distortion. Pumping cannot be classified as distortion, but it results in faulting or sunken slabs. The major causes of distortion are expansive soils, frost-susceptible soils, and foundation settlement.

If not too extensive, some forms of distortion, i.e., settlement, can be remedied by raising the slab to the original grade. One method of doing this is by pumping a high-softening-point undersealing asphalt cement under the slab. Sometimes the pavement surface is so badly distorted that an asphalt concrete overlay is the most economical repair.

Faults

A fault (Figure 9.33) is a difference in elevation between two slabs at a joint or a crack. Faults usually develop from inadequate load transfer between slabs along with consolidation or shrinkage in volume of courses underlying the slabs. They may also be caused by pumping out of the foundation material.

If possible the faulted slabs should be brought back to the original grade with a high-softening-point asphalt underseal. The joint or crack should be cleaned out, sandblasted, blown out with compressed air, and partially filled with a rubber-asphalt joint sealer before undersealing begins. After the low slab is raised to the original grade, the joint or crack is filled with a rubber-asphalt compound. In some cases, it may be necessary to place wedge-type patches for leveling and then place an asphalt overlay. Another acceptable technique is to grind the area between the two slabs to a true, level plane.

Pumping

Pumping is the up and down slab movement under passing loads resulting in the ejection, or pumping, of very wet sand, clay, and / or silt. Pumping can occur along transverse or longitudinal joints and cracks, and along pavement edges (Figure 9.34). Pumping out of the fine material is caused by the presence of free water on or in the subgrade or subbase. The up and down movement of the slab caused by heavy loads forces this water, containing a portion of the subgrade or subbase, up through the crack and on to the surface.

Voids beneath the pavement should be filled with a high-softenig-point asphalt cement underseal. This will also waterproof the subgrade and subbase.

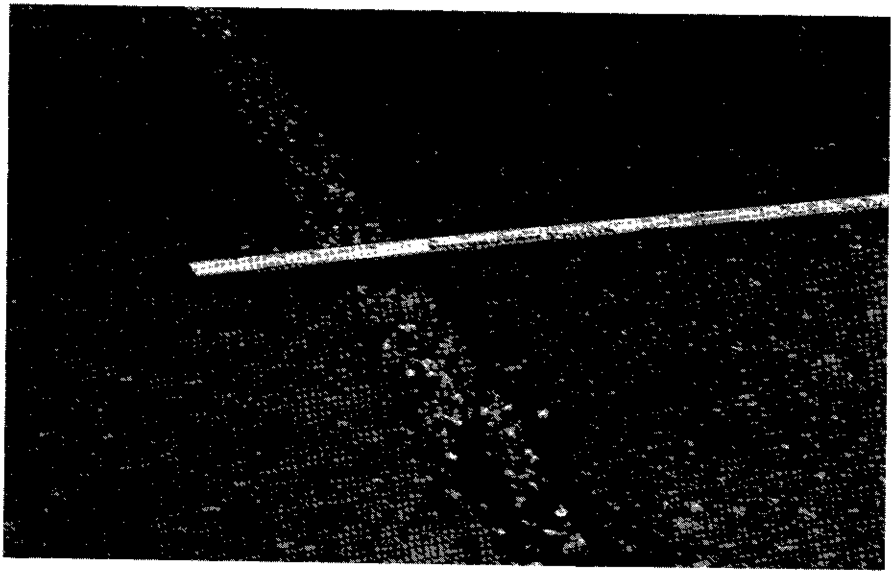

Figure 9.33 **Fault.**

Causes and Repair of Disintegration

Disintegration is the breaking up of a pavement into small, loose fragments. This includes the dislodging of aggregate particles. If not stopped in its early stages, it can progress until the pavement requires complete rebuilding.

Common types of disintegration in portland cement concrete pavements that can be treated with asphalt are:

- Blow-ups
- Spalling
- Scaling

Blow-Ups

A blow-up is the localized buckling or shattering of a rigid type pavement (Figures 9.35 and 9.36) occurring usually at a transverse crack or joint. Most blow-ups occur in hot weather because heat causes the pavement to expand excessively. Pressure builds up until the slabs can no longer resist and they either buckle or shatter, crumbling along the transverse joint or crack.

The damaged portion of the slab is removed by sawing a straight, neat cut with a pavement saw. If necessary, the subbase is leveled, and then primed. A dense-graded hot-mix asphalt is placed and thoroughly compacted. If the area is not large enough for a full-sized roller, vibrating plate rollers or mechanical rammers should be used. The surface of the patch should be finished off flush with the surrounding pavement.

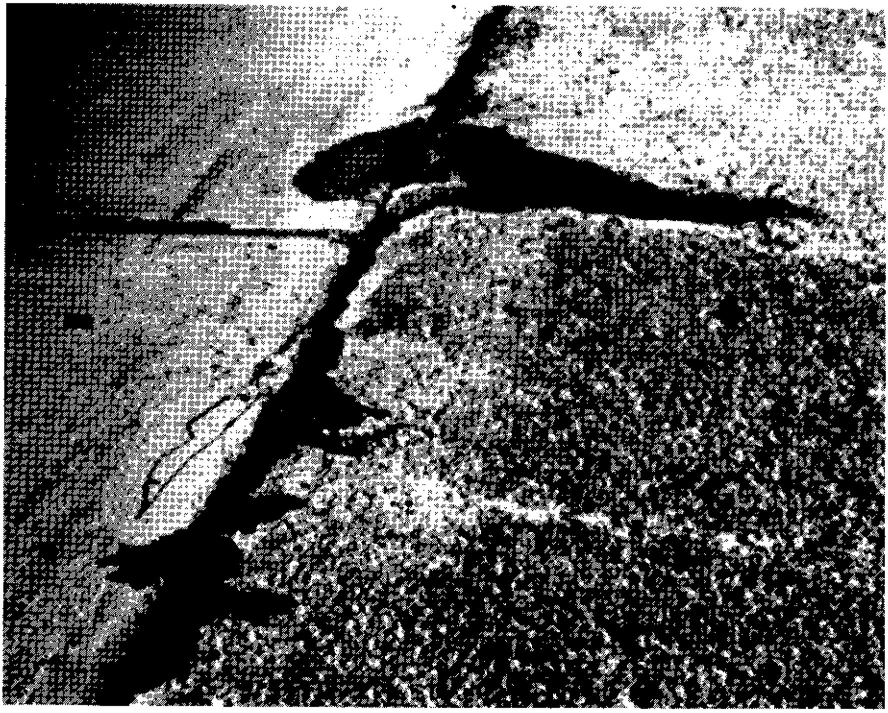

Figure 9.34 Pumping.

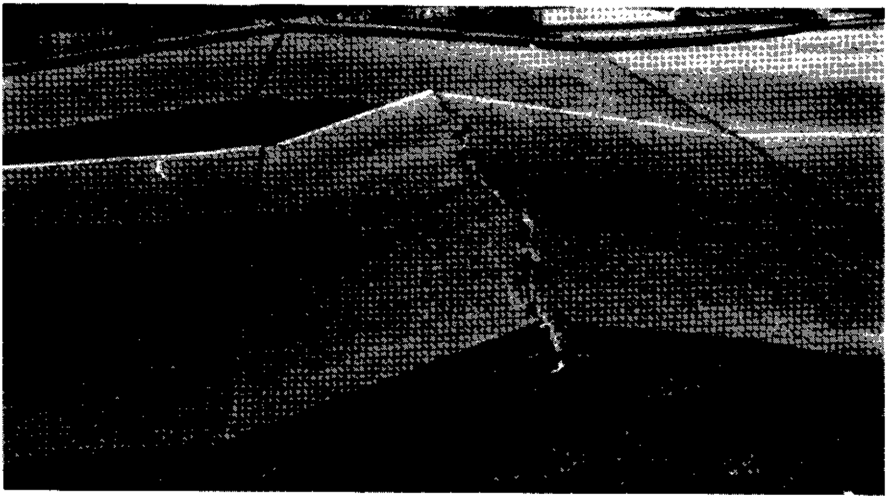

Figure 9.35 Blow-Up (Buckling).

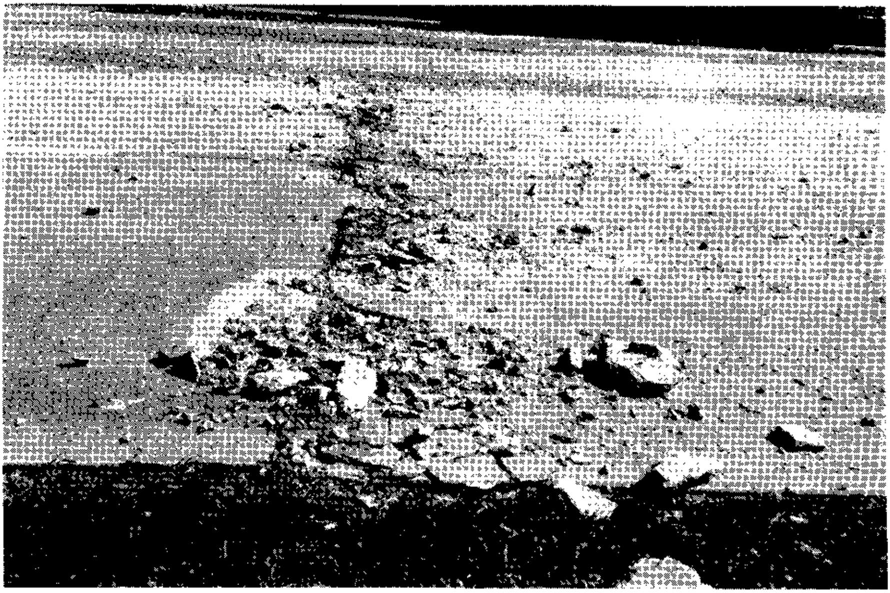

Figure 9.36 Blow-Up (Shattering).

Scaling

Scaling is the peeling away of the surface of the pcc pavement (Figure 9.37). In some cases scaling can progress deeper into the pavement. One of the major causes of scaling is the chemical action of deicing salts. Other causes are improper water-cement ratio, overfinishing, improper air entraining or mixing, unsuitable aggregates, and improper curing.

If the scaled area is 10 mm (⅜ in.) or less in depth, a steel-bristle power broom may be used to dislodge all loose particles. After the area has been thoroughly cleaned, an asphalt-emulsion slurry seal may be applied to restore the surface to the original grade.

If the scaling is extensive and deep, however, the pavement should be overlaid with asphalt concrete.

Spalling

The breaking or chipping of pavement at joints, cracks, or edges is called spalling (Figure 9.38). Usually the resulting fragments have feather edges. This type of pavement disintegration can be caused by a number of different things. Some major causes are hard pieces of gravel becoming lodged in a joint or a crack, improperly installed load transfer devices, improper forming and sawing of joints, and weak mortar.

Repairs may be made by chipping out the spalled area to sound material, squaring the edges, and making the sides as nearly vertical as practical. After the area is thoroughly cleaned of loose material, the hole is given a tack coat and a dense-graded hot-mix asphalt is placed and thoroughly compacted.

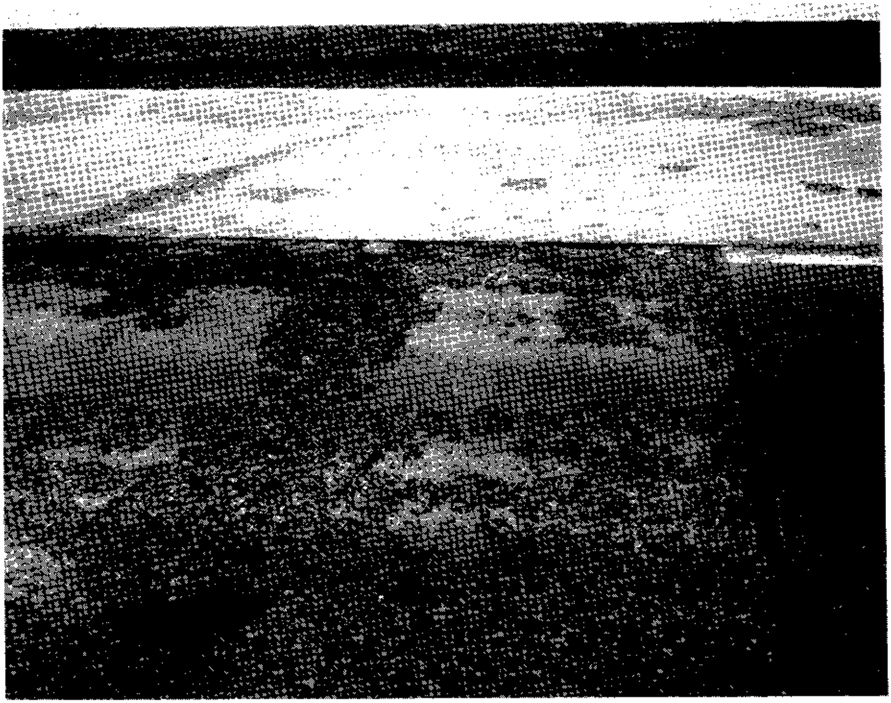

Figure 9.37 Scaling.

Eliminating Skid Hazards

Causes

A number of things can make a pavement slippery when wet. A major cause of slippery portland cement concrete pavements is polished aggregates in the surface. Slipperiness may also develop from surface contamination.

Figure 9.39 shows aggregate particles that have been polished smooth. Some aggregates become polished quickly under traffic. Others, such as some types of gravel, are naturally polished. If these are used in a pavement surface without crushing they become a skid hazard. Polished aggregates are extremely slippery when wet.

Repair

The most effective and economical way to repair a pavement with polished aggregates is to cover the surface with a skid-resistant treatment. A course of hot-mix asphalt containing sharp, angular, non-polishing aggregate should be used. To reduce the detrimental effects of reflection cracking, the overlay should be at least 115 mm (4½ in.) thick.

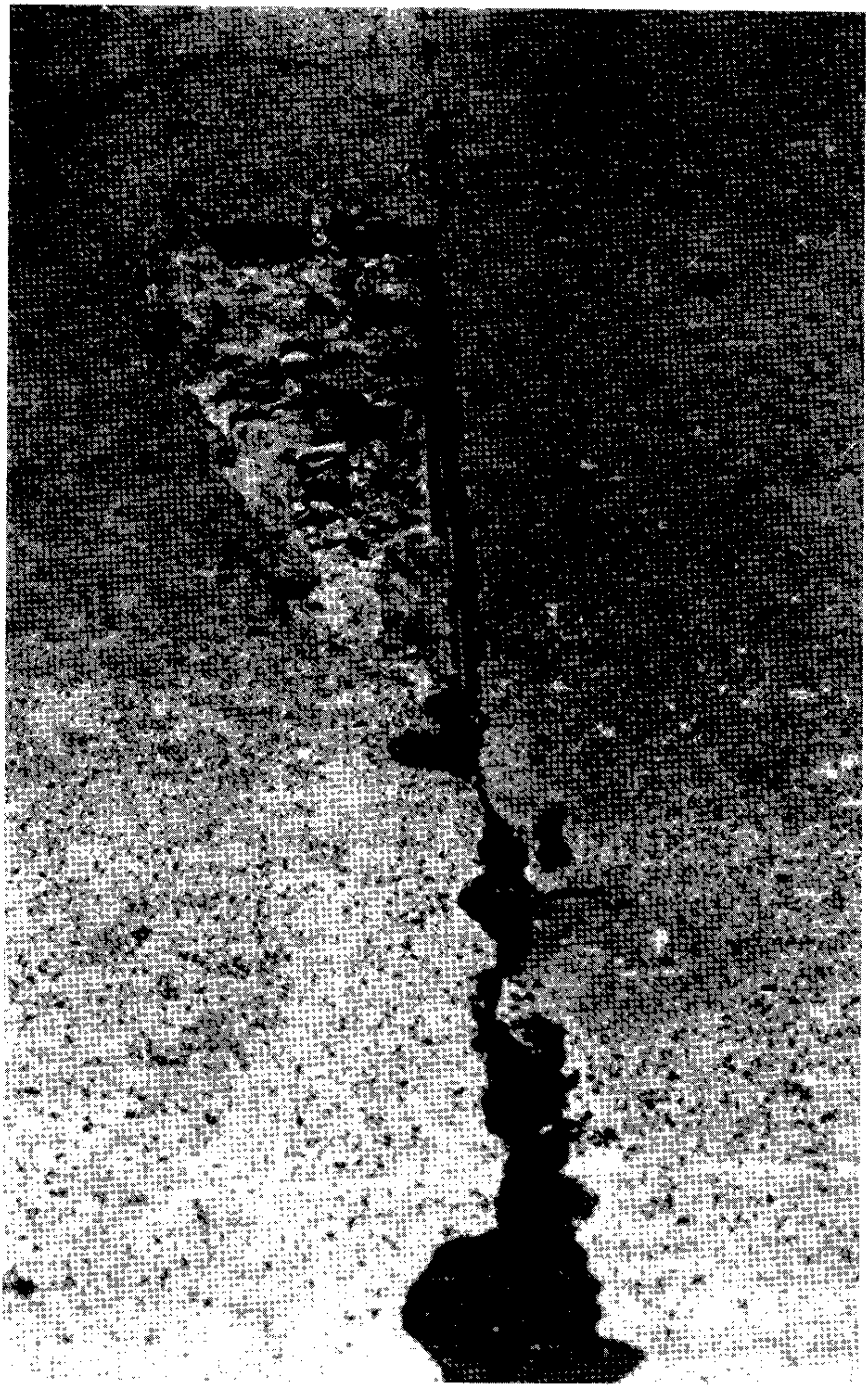

Figure 9.38 Spalling.

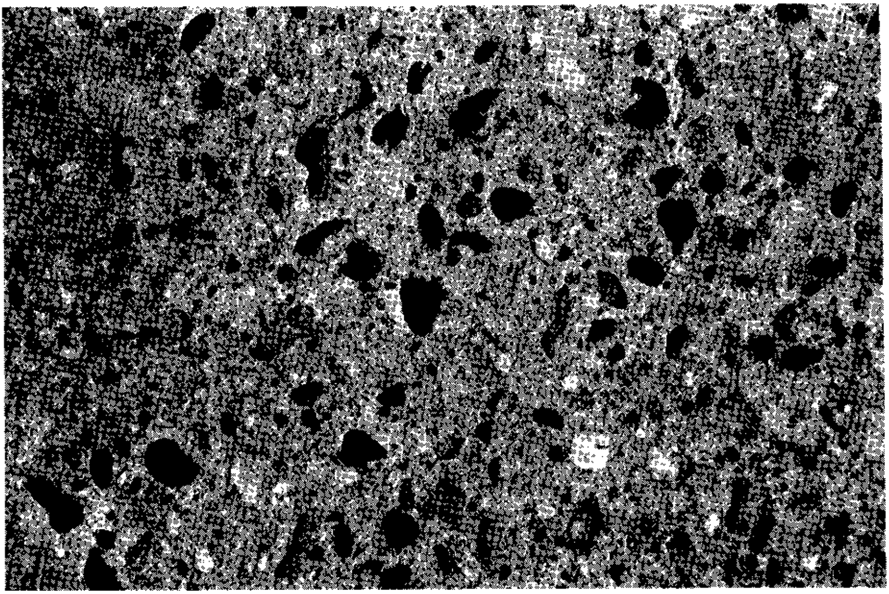

Figure 9.39 Polished Aggregate in the Pavement Surfaces.

9.4 Asphalt in Pavement Rehabilitation

Introduction

As discussed in the previous section, maintenance helps pavements perform satisfactorily during their design life. When this period is exceeded through time or repetitions of traffic, more extensive work is required. It is time for pavement rehabilitation. Pavement rehabilitation is the salvaging, strengthening, and/or modernizing of deficient pavements. A pavement may be rehabilitated by a number of methods, including overlays of varying thickness, recycling of the top layers or complete rebuilding or reconstruction.

Evaluating Pavement Condition

Purpose

To determine if and when a particular pavement requires rehabilitating is one of the functions of a pavement condition survey. Evaluating pavement condition includes measurement of skid resistance, ride quality, structural capacity and surface distress. Surface distress can be in the form of raveling, cracking, rutting or other forms of distortion and, in portland cement concrete pavements, it includes cracking and spalling, pumping and faulting at joints. In evaluating the pavement condition, an assessment is made of the current state of these distress conditions and estimating their future state, or the time when the distress will reach a critical or terminal level; at this time an overlay usually will be required.

Not all measures of pavement condition will reach a terminal level at the same time. For example, a structurally sound asphalt pavement may have a low resistance to skidding, which may be corrected with a thin overlay. A pavement having considerable distress caused by an inadequate structural thickness will require a thicker overlay. In a broader sense, the condition evaluation will assist in setting priorities for overlays or other rehabilitation measures, and help establish the programs to be followed in correcting pavement deficiencies.

Skid Resistance

Many highway agencies find it advantageous to measure skid resistance on all or, at least, selected sections on their road systems on a routine basis, or for the collection of research data. The measurements are made with mechanical devices. In North America the locked-wheel skid trailer [American Society for Testing and Materials (ASTM) Method E 274] is often used. Somewhat less common is the "Mu-Meter" or yaw-mode trailer.

The British Portable Tester also is used, particularly as a research tool.

The skid resistance of pavement surfaces will vary with the amount of traffic, the surface temperature, the surface texture, the thickness of water on the pavement, the vehicle speed and the characteristics of the measuring device. The test is usually made with the pavement wet, and data is often collected at different speeds.

Low skid-values on an otherwise good pavement may indicate the need for some corrective treatment. Recommendations for correcting asphalt pavement surfaces that have skid resistance problems are elsewhere in this chapter.

Ride Quality—Present Serviceability Concept

Most road users know very little about the structure of a road. To them, the measure of adequacy is how well the road is serving them. Starting with this premise, engineers at the AASHO Road Test developed a system for rating the condition of pavement surfaces using what they called the "Present Serviceability Concept." The concept describes present serviceability as the ability of a specific section of pavement to provide, in the opinion of the user, a *smooth and comfortable* ride at that particular time. This ability can be quantified by a value called Present Serviceability Rating (PSR).

Usually, ratings from one (1.0) to five (5.0) are assigned to each section, with the higher numbers indicating the more satisfactory conditions. Low ratings indicate poor pavement condition, primarily a rough pavement, and suggest that a more detailed examination of the surface is required. Some agencies use a rating scale of 0 to 100. These values are comparable to the 1 to 5 rating scale when the lower values are multiplied by 20. Irrespective of the numerical rating scale used, most evaluation systems utilize five general categories of pavement condition—very good, good, fair, poor and very poor.

The measurement of serviceability by mechanical means produces what is known as the Present Serviceability Index (PSI).

The engineer can use the PSR or PSI to decide whether or not to proceed with, or postpone, a more detailed investigation. Present Serviceability may be used as the first step in evaluating the adequacy of a pavement. If a history of PSR or PSI is plotted for sections of roadway that seem to be deteriorating, predictions sometimes may be made as to when corrective measures will be needed.

Condition Rating System

Many engineers believe that Present Serviceability, PSR or PSI, is not sufficient for deciding whether or not a pavement requires an overlay. Several agencies have devised systems for combining measures of pavement distress, in some cases including pavement roughness, into a single rating or index to be used for this purpose. Also, a single index is desirable for use with an automated pavement management system. In the United States several agencies have developed systems that use some type of condition index for use with a pavement management or related system.

A simple condition rating system for asphalt pavements is published by the Asphalt Institute, *A Pavement Rating System for Low-Volume Asphalt Roads* (IS-169). Figure 9.40 shows the rating form for this system. Descriptions and photographs help the rater identify the different types of distress, in addition to giving advice on appropriate corrective actions. The system also includes a scale (Figure 9.41) which can serve as a guide to when overlays are appropriate or when routine maintenance or complete reconstruction are more applicable.

More elaborate systems in use provide for more detailed forms of recording type and degree of distress. The province of Ontario (Canada) has developed guides for relating pavement condition index.

Two guides have been developed in this system. One for asphalt (flexible) pavements (Figure 9.42) and another for portland cement concrete (rigid) pavements (Figure 9.43). In using the system, a pavement rating is first determined using a condition evaluation form, Figure 9.44 or Figure 9.45, to obtain an assessment of the condition of the pavement. Although not done in this case, an index could be calculated by assigning weights to each category of distress shown on the rating forms, as was done for the Asphalt Institute system (Figure 9.40).

When using systems as described above, attention must be given to maintaining consistency of ratings, particularly if more than one rating team is involved. This is done by holding training sessions for raters, using actual pavement sections and photographs or color slides.

Structural Evaluation

Structural evaluation determines the adequacy of a pavement to support traffic without developing appreciable structural distress. The intent of structural evaluation is to determine both the current adequacy of the pavement and to predict its future service life for the traffic using it. When the pavement is found to be inadequate for current or future use the evaluation forms a basis for designing the improvements needed to provide proper service.

A pavement structure may be inadequate for several reasons. The pavement may have simply exceeded its design life—traffic volume and weight may have increased at a rate greater than anticipated in the original design. In such cases, the service life is reduced. Some properties of the pavement's materials may have changed under service conditions—this could have reduced the effectiveness of the material in the pavement structure. Many existing pavements were built before the development of design procedures that take into account the relationships between subgrade support, strength of paving materials, and traffic loadings; thus they may not comply with current structural requirements.

Evaluation techniques for asphalt pavements fall into two general categories. The first, the deflection procedure, involves measuring pavement

ASPHALT PAVEMENT RATING FORM

STREET OR ROUTE ______________ CITY OR COUNTY ______________

LENGTH OF PROJECT ______________ WIDTH ______________

PAVEMENT TYPE ______________ DATE ______________

(Note: A rating of "0" indicates defect does not occur)

DEFECTS		RATING
Transverse Cracks	0-5	______
Longitudinal Cracks	0-5	______
Alligator Cracks	0-10	______
Shrinkage Cracks	0-5	______
Rutting	0-10	______
Corrugations	0-5	______
Raveling	0-5	______
Shoving or Pushing	0-10	______
Pot Holes	0-10	______
Excess Asphalt	0-10	______
Polished Aggregate	0-5	______
Deficient Drainage	0-10	______
Overall Riding Quality (0 is excellent; 10 is very poor)	0-10	______
	Sum of Defects	______

Condition Rating = 100 - Sum of Defects
= 100 - ______________

Condition Rating = ☐

Figure 9.40 Asphalt Pavement Rating Form.

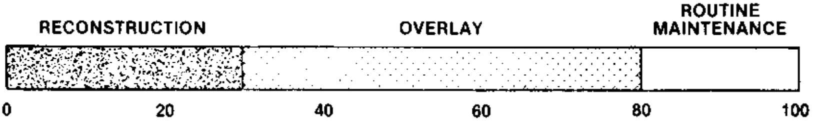

Figure 9.41 Condition Rating as a General Indicator of Type of Maintenance.

Reconstruct or recycle within 2 years.	0-20	Pavement is in poor to very poor condition with extensive severe cracking, alligatoring and channeling. Ridability is poor and the surface is very rough and uneven.
Reconstruct or recycle within 2 - 3 years.	20-30	Pavement is in poor condition with moderate alligatoring and extensive severe cracking and channeling. Ridability is poor and the surface is very rough and uneven.
Overlay, recycle or reconstruct 3 - 4 years.	30-40	Pavement is in poor to fair condition with frequent moderate alligatoring and extensive moderate cracking and channeling. Ridability is poor to fair and surface is moderately rough and uneven.
Reconstruct in 4 - 5 years or resurface within 2 years with extensive leveling.	40-50	Pavement is in poor to fair condition with frequent moderate cracking and channeling, and intermittent moderate alligatoring. Ridability is poor to fair and surface is moderately rough and uneven.
Resurface within 3 years.	50-65	Pavement is in fair condition with intermittent moderate and frequent slight cracking, and with intermittent slight or moderate alligatoring and channeling. Ridability is fair and surface is slightly rough and uneven.
Resurface in 3 - 5 years.	65-80	Pavement is in fairly good condition with frequent slight cracking, slight or very slight channeling and a few areas of slight alligatoring. Ridability is fairly good with intermittent rough and uneven sections.
Normal maintenance only.	80-100	Pavement is in good condition with frequent very slight or slight cracking. Ridability is good with a few slightly rough and uneven sections.
No maintenance required.	90-100	Pavement is in excellent condition with few cracks. Ridability is excellent with few areas of slight distortion.

*Adapted from Transportation Research Board Record No. 700

Figure 9.42 A Guide for the Estimation of Pavement Condition Rating and Priority for Flexible Pavements.

Reconstruct with structural overlay within 2 years.	0-20	Pavement is in very poor condition with severe cracking and faulting. Frequent badly broken and tilted slabs. Ridability is very poor. Extremely rough and uneven throughout.
Reconstruct with structural overlay in 2 - 3 years.	20-40	Pavement is in poor condition with severe cracking and faulting. Intermittent badly broken or tilted slabs. Ridability is poor. Very rough and uneven throughout.
Cut relief joints if necessary. Resurface and underseal within 2 years.	40-50	Pavement is in fair to poor condition with moderate to severe faulting cracks and joints. Ridability is fair to poor and the surface is moderately rough and uneven throughout. Occasional blow ups may occur. Surface moderately polished by traffic.
Cut relief joints if necessary. Resurface and underseal in 2 - 5 years.	50-75	Pavement is in fair condition with moderate faulting at cracks and joints. Ridability is fair and the surface is slightly to moderately rough and uneven throughout. Occasional blow ups may occur. Surface moderately polished by traffic.
Grooving or resurfacing to restore skid resistance if necessary. Otherwise normal maintenance only.	75-90	Pavement is in fair to good condition with slight faulting at cracks & joints Ridability is fair to good with intermittent slightly rough sections. Surface slightly polished by traffic.
Normal maintenance only. Repair joint seals as necessary.	90-100	Pavement is in good condition with little cracking between joints. Intermittent slight faulting at joints. Ridability is good. Skid resistance is satisfactory.

Figure 9.43 A Guide for the Estimation of Pavement Condition Rating and Priority for Rigid Pavements.

deflections and analyzing them with respect to traffic; this procedure has the advantage of directly measuring the responses of the inplace pavement structure to traffic loadings. The second procedure, effective thickness, uses relationships between subgrade strength, pavement structure, and traffic loadings. It is based on the procedure used to design new pavements, except that the structural integrity and thickness of existing pavement components, equated to the thickness of asphalt concrete, must be estimated.

Riding Comfort Rating (At 80 km/h)	Excellent	Good	Fair	Poor	Very Poor
▨	▨	▨	▨	▨	▨

Pavement Distress Manifestations			Severity of Pavement Distress					Density of Pavement Distress (Extent of Occurrence)					Characteristics of Pavement Distresses					
			1	2	3	4	5	6	7	8	9	10	Reflection Cracking	Pavement Edge Crack		Transverse Crack Spacing	Alligator Cracking	
			Very Slight	Slight	Moderate	Severe	Very Severe	Few	Intermittent	Frequent	Extensive	Throughout		Progressive (12 in. from edge) (0.3 m)	Non Progressive (12 in. from edge) (0.3 m)		Alligator Block Size	Distortion of Alligator Areas
								10 %	10 20 %	20 50 %	50 80 %	80 100 %				m	mm	mm
Surface Defects	1 Coarse Aggregate Loss												▨	▨	▨	▨	▨	▨
	2 Ravelling												▨	▨	▨	▨	▨	▨
	3 Flushing												▨	▨	▨	▨	▨	▨
Surface Deformation	4 Rippling												▨	▨	▨	▨	▨	▨
	5 Shoving												▨	▨	▨	▨	▨	▨
	6 Wheel Track Rutting												▨	▨	▨	▨	▨	▨
	7 Distortion												▨	▨	▨	▨	▨	▨
Cracking	Longitudinal Wheel Track	8 Single												▨	▨	▨	▨	▨
		9 Multiple												▨	▨	▨	▨	▨
		10 Alligator												▨	▨	▨		
	Midland	11 Single												▨	▨	▨	▨	▨
		12 Multiple												▨	▨	▨	▨	▨
	Center Line	13 Single												▨	▨	▨	▨	▨
		14 Multiple												▨	▨	▨	▨	▨
		15 Alligator												▨	▨	▨		
	Meander	16 Single												▨	▨	▨	▨	▨
		17 Multiple												▨	▨	▨	▨	▨
	Pavement Edge	18 Single														▨	▨	▨
		19 Multiple														▨	▨	▨
		20 Alligator														▨		
	Transverse	21 Partial											▨	▨	▨		▨	▨
		22 Half											▨	▨	▨		▨	▨
		23 Full											▨	▨	▨		▨	▨
		24 Multiple											▨	▨	▨		▨	▨
		25 Alligator											▨	▨	▨			
	26 Random												▨	▨	▨	▨	▨	▨
	27 Slippage												▨	▨	▨	▨	▨	▨
	28												▨	▨	▨	▨	▨	▨
	29												▨	▨	▨	▨	▨	▨
▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨	▨
Maintenance Patching		Spray	▨	▨	▨	▨	▨						▨	▨	▨	▨	▨	▨
		Skin	▨	▨	▨	▨	▨						▨	▨	▨	▨	▨	▨
		Hot Mix	▨	▨	▨	▨	▨						▨	▨	▨	▨	▨	▨
Additional Remarks																		

Figure 9.44 Flexible Pavement Condition Evaluation Form.

The structural evaluation procedure for portland cement concrete pavements also includes two methods of design. The first, or deflection procedure, includes measurements of differential deflection, such as those caused by load transfer at joints. The second, the effective thickness procedure, is similar to the second procedure used for evaluating asphalt pavements.

Information for performing structural evaluations are contained in Asphalt Overlays for Highway and Street Rehabilitation, MS-17, the Asphalt Institute. Methods include deflection procedures (i.e., Benkelman Beam, Dynaflect, Road Rater, falling weight deflectometer, etc.) or various effective thickness approaches (see Chapter 11).

Riding Comfort Rating (At 80 km/h)	Excellent	Good	Fair	Poor	Very Poor

Pavement Distress Manifestations			Severity of Pavement Distress					Density of Pavement Distress (% of Occurrence)					Characteristics of Pavement Distress			
			Very Slight	Slight	Moderate	Severe	Very Severe	Few	Intermittent	Frequent	Extensive	Throughout	Reflection Crack	Transverse Crack Spacing	Joint or Crack With Pumping	Joint or Crack With Debris
								< 10%	10 20 %	20 50 %	50 80 %	80 100 %		(m)		
Surface Defects	Polishing												▨	▨	▨	▨
	Loss of Coarse Aggregates												▨	▨	▨	▨
	Pot Hole												▨	▨	▨	▨
	Scaling												▨	▨	▨	▨
	Raveling												▨	▨	▨	▨
Surface Deformation	Faulting												▨	▨	▨	▨
	Settlement												▨	▨	▨	▨
Joint Deficiencies	Joint Creeping												▨	▨	▨	▨
	Joint Sealant Loss												▨	▨	▨	
	Joint Spalling												▨	▨		
	Joint Failures (Blow Up Etc.)		▨	▨	▨								▨	▨		
Cracking	Longitudinal													▨		
	Meandering													▨		
	Corner													▨		
	D													▨		
	Transverse	Single														
		Multiple														
	Diagonal													▨		
	Edge Crescent													▨		
	Miscellaneous Cracks													▨		
														▨		
Miscellaneous Distresses	Lane Separation													▨		
	Slab Warping												▨	▨	▨	▨
	Wheel Track Wear												▨	▨	▨	▨
	Others												▨	▨	▨	▨
Maintenance	Full Width Joint Repair		▨	▨	▨	▨	▨						▨	▨	▨	▨
	Full Depth Relief Joint		▨	▨	▨	▨	▨						▨	▨	▨	▨
	Precast Slab		▨	▨	▨	▨	▨						▨	▨	▨	▨
	Cold Mix Patching		▨	▨	▨	▨	▨						▨	▨	▨	▨
	Full Width HL Patch		▨	▨	▨	▨	▨						▨	▨	▨	▨
Additional Remarks																

Figure 9.45 Rigid Pavement Condition Evaluation Form.

Asphalt Overlays

Types of Overlays

Asphalt overlays may be used to correct both surface deficiencies (raveling, roughness, slipperiness) and structural deficiencies. Surface deficiencies in asphalt pavements usually are corrected by thin resurfacings, but structural deficiencies require overlays designed on factors such as pavement properties and traffic loadings.

Thin overlays usually range from 25 mm (1 in.) up to 50 mm (2 in.) thick, using a fine-grained [9.5 or 12.5 mm (⅜ in. or ½ in. top size aggregate)] dense mix.

An open-graded wearing surface consists of a high-void, hot or cold asphalt mix that is characterized by a large percentage of one-sized coarse aggregate that will ensure a high void (15 to 25 percent) content mix. Its major advantage is its ability to provide a skid-resistant surface that will minimize hydroplaning. It is generally made with a 9.5 mm (⅜ in.) maximum size aggregate and asphalt cement, (AC-10 to AC-40, AR-4000 to AR-16000 or 85-100 to 40-50 pen.). Prior to construction, all structural defects should be corrected and cracks greater than 3 mm (⅛ in.) in width should be sealed.

Structural deficiencies require additional strength from overlays based on pavement properties and traffic. The approach used for the structural design of an overlay may be categorized as component analysis, or deflection analysis as discussed earlier.

Correcting Surface Deficiencies with Overlays

The major reasons for overlaying otherwise adequate pavements are excessive permeability, surface raveling, surface roughness, and surfaces with low skid resistance.

Permeability and Raveling—The causes of excessive permeability and raveling usually are the same. These are construction during wet or cold weather, mixtures that lack sufficient asphalt, or pavements that have not been compacted to proper density. Any of those causes may affect pavement strength, but not necessarily to the point where structural distress will develop. The requirements for effective corrective treatment of excessive permeability and raveling are the same: a surface treatment or overlay that also seals the surface.

Roughness—Surface roughness may be caused by settlement, raveling, nonuniform wear, corrugations and similar irregularities. Local structural distress should be corrected, followed by local or general leveling of minimum thickness to ensure a smooth riding surface. Leveling should be followed by a surface treatment or hot-mix asphalt overlay.

A distorted cross-section may result in a poor riding surface on an otherwise satisfactory pavement. Correction of this deficiency involves construction of leveling wedges followed by an overlay.

In many cases riding quality can be restored by planing or cold-milling the rough or distorted surface of the pavement. If it is determined that a structural overlay is not required, a thin overlay should be applied to replace the material removed.

Slippery Surface—Corrective design for a slippery surface requires a skid-resistant surfacing material. Properly designed sand-asphalt or fine-graded asphalt concrete made with hard, polish-resistant aggregate is required. In some cases, surface treatments and slurry seals can also be used effectively. If hydroplaning is a problem an open-graded wearing or friction course can be applied. Open-graded courses also can be used to reduce traffic noise.

If excess asphalt (bleeding) exists on the surface, the condition must be corrected before the overlay is placed. In many cases, bleeding can be corrected by repeated applications of hot sand, slag or rock screenings to blot up the excess asphalt.

A pavement planing machine will remove the excess asphalt. Or in rare instances of heavily over-asphalted surfaces, the surfaces should be completely removed and replaced.

Renewing Overlays

The surface-renewing overlay should consist of material which can be constructed in a thin layer. It must fill surface voids and provide an impervious, skid-resistant surface. It must also be sufficiently resistant to traffic abrasion to provide an economical service life. Materials meeting these requirements are asphalt concrete having small maximum particle size or hot-mix sand-asphalt mixtures.

Open-graded asphalt friction courses can be used to prevent hydroplaning, to reduce tire splash and spray, and to provide skid-resistant surfaces. However, surfaces on which they are placed must be impervious and reasonably smooth. Recommendations on the design and construction of open-graded asphalt friction courses are given in *Open-Graded Asphalt Friction Courses,* Construction Leaflet No. 10 (CL-10), published by the Asphalt Institute.

Preparing Pavements for Overlay

The overlay thickness is designed to correct a below-average pavement condition, but not to provide the extra structural strength needed for localized weak areas. If the overlay thickness is based on the weakest condition in the section it would be over-designed for the rest of the section and thus be needlessly costly. Therefore, the weaker areas if possible should be corrected to provide a uniform foundation for the overlay. Careful and correct preparation of the existing pavement, prior to construction of either smoothing or strengthening overlays, is essential to good construction and to maximum overlay performance.

The correct preparation of existing pavements for overlays varies with the pavement type and, for this reason, each is discussed separately below.

All weak areas should be repaired with proper patches. The extent of weak areas (those with excessive deflection) can be determined with a Benkelman Beam or similar equipment. Deflection readings should be taken at a number of locations sufficient to outline the whole area of excessive deflection before repairs are made. Areas of excessive deflection may be estimated by comparing deflection in the distressed area with the average deflection in areas that are performing well. Structural patches should be designed and constructed with Full-Depth asphalt concrete to ensure strength equal to or exceeding that of the surrounding pavement structure. Carefully-placed and adequately compacted patches will produce a uniform supporting layer for the overlay, ensuring good performance.

When the surface is distorted, the construction of leveling courses or leveling wedges may be required to restore proper line and cross-section. Cold planing may also be used to establish a true grade and profile. When thin surfacing courses (such as open-graded asphalt friction courses) are to be used it is especially important that prior correction of the surface contour be made. Leveling wedges are described in detail in Chapter 6.

Additional Design Considerations for Structural Overlays

In constructing overlays, the problem of reflection cracking must be considered. Although the overlay thickness might be adequate to correct standard deficiencies, reflection cracking can adversely affect the structural integrity and riding quality of the rehabilitated pavement and require increased maintenance. Reflection cracking can be defined as the migration of subsurface crack pattern into and subsequently through an overlay. It develops from movement of the subsurface layers and / or vertical stresses. It is believed to be caused by the transfer of high horizontal stresses to the overlay at cracks in the underlying pavement (see Figure 9.46). Some type of crack relief system may be required to minimize the transmission of reflection cracks through the overlay.

Overlays on Portland Cement Concrete

Reflection cracks occur most frequently when asphalt mixes are placed on portland cement concrete and soil-cement or lime-fly ash bases. This is particularly prevalent when cracks in the old, overlaid pavement were not properly repaired. There is no typical pattern for these cracks; they mostly reproduce those present in the original pavement.

Many systems have been devised and tested for minimizing reflection cracking. There are several methods currently showing some promise for controlling reflection cracking:

- Cracking and Seating of pcc
- Rubblizing
- Crack-Relief Layer Between pcc and Overlay
- Asphalt-Rubber Interlayer
- Paving Fabric Interlayer
- Sawcut and Sealing

The cracking and seating process consists of controlled inducing of cracks into deteriorated portland cement concrete (pcc) slabs in order to reduce the effective slab length and seating the pieces onto the subgrade by rolling. The procedural steps of the process are:

1. Crack pcc slabs
2. Seat cracked pieces
3. Remove and patch soft areas

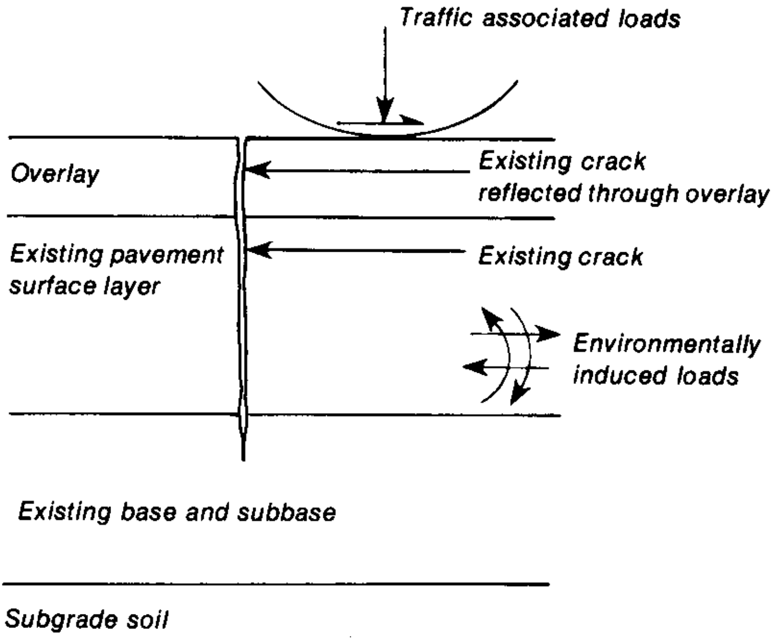
Traffic associated loads
Overlay
Existing crack reflected through overlay
Existing pavement surface layer
Existing crack
Environmentally induced loads
Existing base and subbase
Subgrade soil

Figure 9.46 Schematic Diagram of Reflection Cracking.

4. Sweep
5. Apply tack coat
6. Place asphalt leveling course/overlay

Cracking and seating of deteriorated pcc prior to overlay is especially effective for non-reinforced plain-jointed slabs. For heavily-reinforced slabs, cracking alone is not always sufficient. The bond between the steel reinforcement and the concrete has to be "broken," hence the name "break and seat." Another alternative showing much promise involves completely "rubblizing" the pcc in place, reducing it to a 75mm (3-inch) minus sieve size material, thus creating an effective granular base.

Leading candidates for crack and seat are plain-jointed concrete pavements with long joint spacing. The crack and seat procedure is a rehabilitation technique that has proven effective in controlling reflective cracking in overlays. The method has been used for over 30 years by various states.

For more information on these processes refer to the Asphalt Institute Technical Bulletins TB-1, Cracking & Seating, and TB-3, Rubblizing.

The states of Arkansas and Tennessee pioneered the use of the crack-relief layer. It has been used primarily over deteriorated portland cement concrete pavements, both plain-jointed and continuously reinforced. It consists of a 90mm (3-½ in.) thickness of coarse, open-graded hot mix containing 25-35 percent interconnected voids, and made up of 100 percent crushed large size aggregate material. It is placed as the first course of an overlay system. Because of the large amount of interconnecting voids, this crack-relief layer provides a medium through which differential movements of the underlying slabs are not readily transmitted. Following placement of the crack-relief layer, a dense-graded asphalt concrete mix 75 mm (3 in.) thick is placed in two lifts. This system has proven to be very effective. Since the minimum thickness of this overlay is 165mm (6.5 in.) it may not be applicable to all roadways, particularly those having curb and gutter sections or bridge clearances.

A thin rubber-asphalt surface treatment is also being used as an interlayer. There are two processes, both developed in Arizona. Each consists of a thin rubber-asphalt surface treatment of about 2.7 litre/m^2 (0.6 gal/yd^2) or rubber-asphalt binder. It is followed immediately by about 22 kg/m^2 (40 lb/yd^2) of 12.5 or 9.5 mm (½ or ⅜ in.) nominal size cover aggregate. In the first process, the binder is produced using 25 percent by weight ground tire-tread rubber (95 percent passing 1.18 mm (No. 16 sieve) and 75 percent asphalt cement, AC-5 (AR-2000). It is heated and reacted in a distributor to a temperature between 177° to 204°C (350° to 400°F) and then cut back with 5 to 7 percent kerosene by volume. In the second process, the binder is produced using 20 percent rubber (consisting of 60 percent vulcanized rubber and 40 percent devulcanized rubber) and 80 percent asphalt cement heated and cut back as in the first process. Both of these surface treatment processes have been reported to be effective in reducing fatigue-crack reflection, but they are susceptible to the reflection of thermal or shrinkage cracks.

Another method being used is the fabric system. This system consists of a fabric used as an interlayer between a pavement and an asphalt overlay or surface treatment. The fabric is manufactured and marketed by a number of suppliers. It is produced from polyester or polypropylene as a woven or nonwoven material. After pavement preparation, which may include crack-sealing, leveling course, heater-planing, or cold grinding, a binder or tack coat is applied to the old pavement surface. This is necessary to adequately bond the fabric and allow for absorption of asphalt. Asphalt cements or rapid-set (RS or CRS) emulsified asphalts are used depending on the particular manufacturers recommendations. The spray rate is adjusted to provide the necessary residual asphalt application. Adequate cure time for emulsions is allowed prior to fabric laydown. Although fabric can be laid manually, for large jobs special laydown equipment is employed. It is possible for paving operations to follow immediately behind fabric placement. When installed over old portland cement concrete slabs having differential movement at joints, it appears a fabric containing asphalt may continue to act as a water-proofing membrane but will not stop reflection cracking if the movement is appreciable. Also, the presence of fabric interlayers may pose future problems when pavements are considered for recycling.

Sawcut and sealing of an asphalt concrete overlay is another technique to reduce the detrimental effects of reflective cracking at the joints and extend the service life of an overlay. The sawcut and seal technique establishes a joint in the overlay directly above the existing portland cement concrete joint. It is then sealed. The procedural steps in the process are:

1. Locate and reference existing joints in the pcc slab
2. Place overlay
3. Sawcut directly above referenced joint
4. Clean and dry sawcut
5. Seal sawcut

Detailed information on this procedure is available in Asphalt Institute Technical Bulletin TB-2, Sawcut and Seal.

Chapter 10
Soils and Drainage for Asphalt Pavements

This chapter is not a course in soil mechanics or drainage. Rather, the reader should already be versed in these areas. This chapter discusses the subgrade classification methods and tests used for evaluating foundation soils for highway, airport and other pavement engineering purposes. Knowledge of these classification methods and strength evaluation tests are invaluable to the design and construction of asphalt pavements.

Likewise, drainage of asphalt pavements is also an important aspect in asphalt pavement design and construction. The latter part of this chapter reviews some of the key concerns of drainage as related to pavements.

SECTION 10.1 **Soil-Subgrade Classification Systems**
☐ Introduction
☐ AASHTO Classification System
☐ Unified Soil Classification System

SECTION 10.2 **Soil-Subgrade Strength Evaluation Methods**
☐ Introduction
☐ Plate Bearing Test
☐ California Bearing Ratio Method
☐ Resistance Value Method
☐ Resilient Modulus Method

SECTION 10.3 **Drainage of Asphalt Pavement Structures**
☐ Introduction
☐ Surface Drainage
☐ Subsurface Drainage
☐ Construction

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2. *Drainage of Asphalt Pavement Structures, MS-15,* Asphalt Institute.

10.1 Soil-Subgrade Classification Systems

Introduction

Objective for Classifying Subgrade

The objective behind using any soil classification system for road and airport construction is to be able to predict subgrade performance of a given soil on the basis of a few simple tests performed on soil in a disturbed condition. On the basis of these test results and their correlation with field experience, soil may then be correctly identified and placed into a group of soils, all of which have similar characteristics and properties.

Grouping soils into some classification system is not the final objective. That is, the classifying of a soil should not be regarded as an end in itself, but as a tool to understand soil action and behavior.

Classification Systems

Two classification systems will be described:

1. AASHTO Classification System for Soils, and
2. Unified Soil Classification System.

The principal soil classification system used in the United States by highway engineers is the American Association of State Highway and Transportation Officials (AASHTO) system. Originally this system was developed by the U.S. Bureau of Public Roads about 1928. It was revised several times by the Bureau of Public Roads up to 1942. An extensive revision was made by a committee of highway engineers for the Highway Research Board in 1945. The 1945 version formed the basis for the present AASHTO system.

The Unified Soil Classification System is based on the Airfield Classification System developed by Professor A. Casagrande of Harvard University during World War II. The system was modified slightly and became the Department of the Army Unified Soil Classification System. In turn, this system was slightly modified and adopted by the Bureau of Reclamation and the Department of Defense.

AASHTO Classification System

General

According to the AASHTO system, soils having approximately the same general load-carrying capacity and service characteristics are grouped together to form seven basic groups (A-1, A-2, A-3, A-4, A-5, A-6, and A-7). In general, the best highway subgrade soils are classified as A-1. Poorer soils are rated in numerical order with the A-7 group being the poorest. The exception is the A-3 group. Soils classified A-3 are better subgrades than soils in the A-2 group. From this, it can generally be assumed for a given set of conditions, the thickness of the pavement must progressively increase as the soil classification group increases from A-1 to A-7, except as noted.

The revised system currently in use employs the seven basic groups, but some have been subdivided, making a total of 12 groups and subgroups (Table 10.1). A group index is used not to place a soil in a specified group, but as a means of evaluating soils as subgrade materials within their groups. Group index is a function of the liquid limit, plasticity index, and the amount of material passing the 75 μm (No. 200) sieve. It may be calculated by:

$$\begin{aligned} \text{Group Index (GI)} &= (F - 35)\,[0.2 + 0.005\,(LL - 40)] \\ &\quad + 0.01\,(F - 15)\,(PI - 10) \end{aligned}$$

where F = percentage passing 75 μm (No. 200) sieve, expressed as a whole number. [This percentage is based on the material passing the 75 mm (3 in.) sieve.]

LL = Liquid Limit

PI = Plasticity Index

When the calculated group index is negative, the group index is reported as zero. The group index should be reported to the nearest whole number.

Table 10.1 Classification of Soils and Soil-Aggregate Mixtures (With Suggested Subgroups)

General Classifications	Granular Materials (35% or less passing 75 μm)							Silt-Clay Materials (More than 35% passing 75 μm)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve Analysis, Percent Passing											
2.00 mm (No. 10)	50 max	——	——	——	——	——	——	——	——	——	——
425 μm (No. 40)	30 max	50 max	51 min	——	——	——	——	——	——	——	——
75 μm (No. 200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of Fraction Passing 425 μm (No. 40)											
Liquid Limit	——		——	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity Index	6 max		N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min[1]
Usual Types of Significant Constituent Materials	Stone Fragments Gravel and Sand		Fine Sand	Silty or Clayey Gravel and Sand				Silty Soils		Clayey Soils	
General Rating as Subgrade	Excellent to Good							Fair to Poor			

[1] Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 is greater than LL minus 30 (See Figure 10.1).

Classification

The classification of a specific soil is based upon the results of tests made in accordance with standard methods of soil testing, such as:

1. Sieve analysis of fine and coarse aggregate (AASHTO T 27, ASTM C 136);
2. Mechanical analysis of soils (AASHTO T 88, ASTM D 422);
3. Liquid limit of soils (AASHTO T 89, ASTM D 4318); and
4. Plastic limit and plasticity index of soils (AASHTO T 90, ASTM D 4318).

After necessary laboratory tests have been performed, the classification for a given material can normally be made without great difficulty. Nevertheless, the individual making the classification should be aware of several terms that frequently arise. Additionally, an understanding of certain peculiarities and general performance characteristics of the various soil types is essential to their proper use as highway materials.

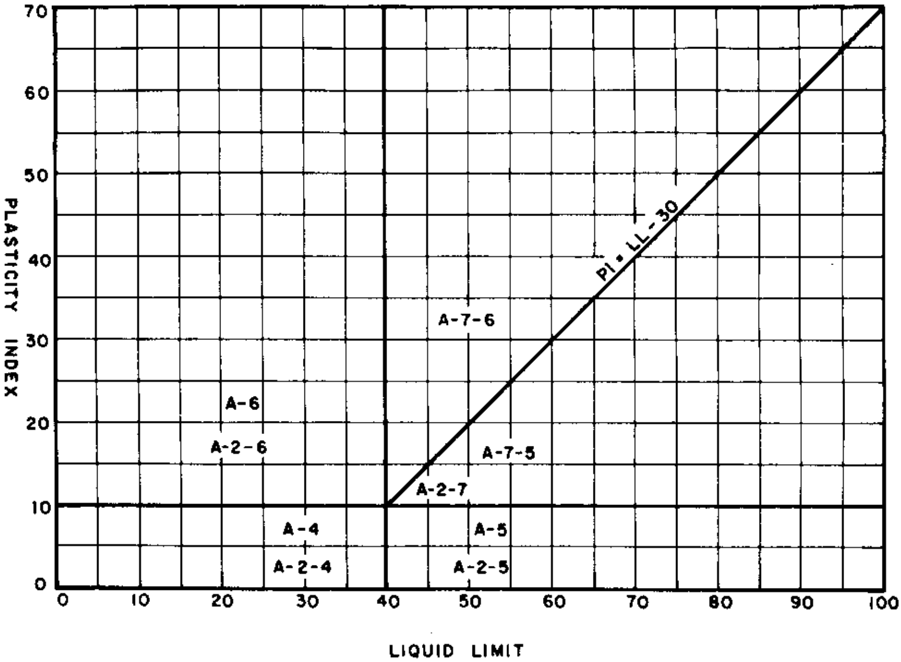
According to the AASHTO system, soils are divided into two major categories: granular materials with 35 percent or less passing the 75 μm (No. 200) sieve, and silt-clay materials with more than 35 percent passing the 75 μm (No. 200) sieve. In addition, the following five soil fractions are recognized and often used in word descriptions of a material.

1. *Boulders*—material retained on 75 mm (3 in.) sieve. They should be excluded from the portion of a sample to which the classification is applied, but the percentage of such material should be recorded.
2. *Gravel*—material passing 75 mm (3 in.) sieve and retained on the 2.00 mm (No. 10) sieve.
3. *Coarse sand*—material passing 2.00 mm (No. 10) sieve and retained on the 425 μm (No. 40) sieve.
4. *Fine sand*—material passing the 425 μm (No. 40) sieve and retained on the 75 μm (No. 200) sieve.
5. *Combined silt and clay*—material passing the 75 μm (No. 200) sieve.

The word "silty" is applied to a fine material having a plasticity index (PI) of 10 or less, and the term "clayey" is applied to fine material having a PI of more than 10.

Classification Procedure

The final soil classification is obtained by using the results from the required tests mentioned earlier and determining the group index. Using the test data, proceed from the left to right in Table 10.1 and the correct group will be found by the process of elimination. Classifying soils may be done by using Figure 10.1. All limiting test values are shown as whole numbers. If fractional numbers appear on test reports, convert to the nearest whole number for classification purposes.



[Note: A-2 soil contains less than 35% finer than 75 μm (No. 200) sieve.]

Figure 10.1 Liquid Limit and Plasticity Index Ranges for A-2, A-4, A-5, A-6, A-7 subgroups.

Group index values should always be shown in parentheses after the group symbol, such as A-2-6(3), A-4(5), etc. For example, classify the soil from the following information:

Percent passing

2.00 mm (No. 10) sieve	=	75
425 μm (No. 40) sieve	=	75
75 μm (No. 200) sieve	=	28
Liquid limit	=	41
Plastic limit	=	34

Solution:

PI = LL - PL = 41 - 34 = 7

GI = (-1.435) + (-0.39) = -1.825. Therefore the group index is reported as zero.

Beginning at the left of Table 10.1, the A-1 and A-3 classifications are quickly eliminated on the basis of the sieve analysis. Based on the characteristics of the fraction passing the 425 μm (No. 40) sieve, the soil will be classified in the A-2-5 column. Since this classification has no group index value, the group index information is omitted. The classification is simply A-2-5.

Unified Soil Classification System

General

The Unified Soil Classification System is based on textural characteristics for those soils with such a small amount of fines that the fines do not affect soil behavior. It is based primarily on the characteristics that determine how a soil will behave when used as a construction material. These characteristics are evaluated by plotting the plasticity index (PI) vs. the liquid limit (LL) on a standard plasticity chart (Figure 10.2). The position of the plotted points yields information from which to predict soil behavior as an engineering construction material. The following properties form the basis of soil identification:

1. Percentages of gravel, sand, and fines fraction passing 75μm (No. 200);
2. Shape of the grain-size distribution curve; and
3. Plasticity and compressibility characteristics.

A soil is given a descriptive name and letter symbol to indicate its principal characteristics.

Four soil fractions, cobbles, gravel, sand and fines are recognized in this system to designate the size ranges of soil fractions (Table 10.2). The minus 75 μm (No. 200) sieve material is silt if nonplastic and LL and PI plot below the A line of Figure 10.2; it is clay if plastic and the LL and PI plot above the A line. This holds true for inorganic silts and clays and organic silts, but not for organic clays since they plot below the A line.

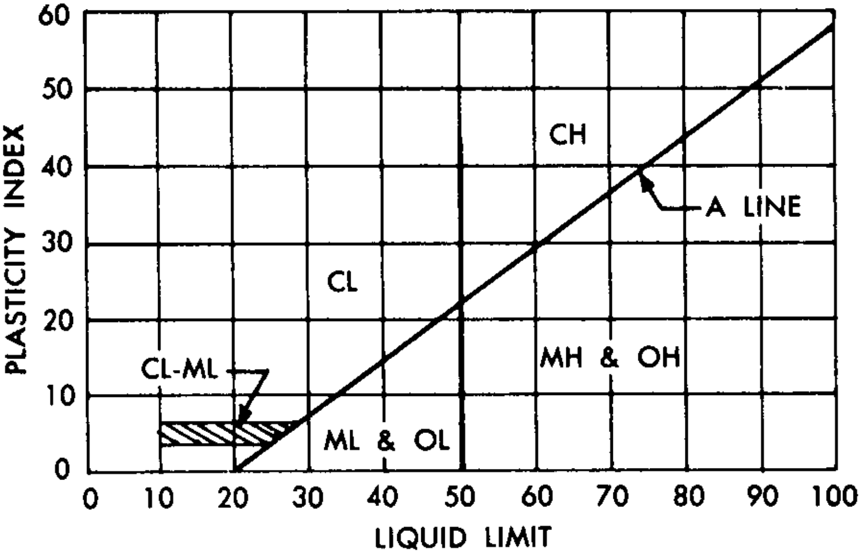


Figure 10.2 Plasticity Chart for Unified Soil Classification.

Table 10.2 Unified Soil Classification Size Ranges of Soil Fractions

Component	Size Range	
Cobbles	above 75 mm	(above 3 in.)
Gravel	75 mm to 4.75 mm	(3 in. to No. 4 sieve)
Coarse gravel	75 mm to 19.0 mm	(3 in to ¾ in.)
Fine gravel	19.0 mm to 4.75 mm	(¾ in. to No. 4 sieve)
Sand	4.75 mm to 75 μm	(No. 4 to No. 200)
Coarse sand	4.75 mm to 2.00 mm	(No. 4 to No. 10)
Medium sand	2.00 mm to 425 μm	(No. 10 to No. 40)
Fine sand	425 μm to 75 μm	(No. 40 to No. 200)
Fines (silt or clay)	below 75 μm	(below No. 200)

Soil Groups

The Unified Soil Classification System places soils into three divisions:

1. Coarse-grained,
2. Fine-grained, and
3. Highly organic

Coarse-grained soils are those having 50 percent or less material passing 75 μm (No. 200) sieve. Fine-grained soils are those having more than 50 percent passing the 75 μm (No. 200) sieve. Highly organic soils can generally be identified by visual examination.

This system recognizes 15 soil groups and uses names and easily remembered letter symbols to distinguish between these groups. Letter symbols (Table 10.3) are derived either from the terms descriptive of the soil fractions, the relative value of the liquid limit (high or low), or relative gradation (well-graded or poorly graded). These symbols are combined to form the group symbols that correspond to the names of typical soils (Table 10.4). They are also used in combination in describing borderline soils.

Coarse-grained soils are subdivided into gravels and gravelly soils (G), and sands and sandy soils (S). Gravels are those having the greater portion of the material retained on the 4.75 mm (No. 4) sieve. Sands are those having the greater portion passing the 4.75 mm (No. 4) sieve and retained on the 75 μm (No. 200) sieve.

Fine-grained soils are subdivided into silts (M for the Swedish terms mo and mjala or flour) and clays (C) based on their liquid limit and plasticity index. Organic soils (O) are also included in this fraction. Silts are fine-grained soils that have LL and PI that plot below the A line of Figure 10.2; clays plot above the A line. Organic clays are the exception to the rule, as the LL and PI plot below the A line. Silt, clay, and organic fractions are further subdivided on the basis of relatively low (L) or high (H) liquid limits. The arbitrary dividing line between the low and high liquid limits has been set at 50.

Highly organic soils are in one group (Pt). They are usually very compressible and have undesirable construction characteristics. Typical organic soils are peat, humus, and swamp soils with a highly organic texture.

Table 10.3 Unified Soil Classification Symbols for Components, Gradation, and Liquid Limit

Component	Symbol
Boulders	None
Cobbles	None
Gravel	G
Sand	S
Silt	M
Clay	C
Organic	O
Peat	Pt
Well-graded	W
Poorly-graded	P
High L.L.	H
Low L.L.	L

Common components of these soils are particles of leaves, grass, branches, or other fibrous vegetable matter.

Field Classification

The Unified Soil Classification System is designed so that most soils may be classified into at least three primary groups by visual inspection and simple field tests. With experience, classification into the subdivisions can successfully be made by visual inspection. Since most soils are given at least a preliminary classification in the field, the ability to field classify by the unified system is a great advantage. When required, a more positive identification may be made by laboratory tests. Field and laboratory test methods are similar in many respects.

Tests used in field identification are the dilatancy or shake test, dry strength, and toughness or consistency near the plastic limit. Procedures for performing these tests are found below Table 10.4.

Classification consists of a process of elimination, beginning at the left side of Table 10.4 and working to the right until the proper group is obtained.

Laboratory Classification

The same descriptive information required for field classification by the Unified System is also necessary for laboratory classification. The field

Table 10.4 Unified Soil Classification (Including Identification and Description)

Major Divisions			Group Symbols	Typical Names	Field Identification Procedures Excluding particles larger than 75 mm (3 in.) and basing fractions on estimated weights.		
1	2		3	4	5		
Coarse-grained Soils More than half of material is larger than 75 µm (No. 200) sieve size The 75 µm (No. 200) sieve size is about the smallest particle visible to the naked eye	Gravels More than half of coarse fraction is larger than 4.75 mm (No. 4) sieve size For visual classification, the 6.3 mm (¼ in.) size may be used as equivalent to the 4.75 mm (No. 4) sieve size	Clean Gravels (Little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.		
			GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines.	Predominantly one size or a range of sizes with some intermediate sizes missing.		
		Gravels with Fines (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.	Nonplastic fines or fines with low plasticity. (for identification procedures see ML below)		
			GC	Clayey gravels, gravel-sand-clay mixtures.	Plastic fines (for identification procedures see CL below).		
	Sands More than half of coarse fraction is smaller than 4.75 mm (No. 4) sieve size	Clean Sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines.	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.		
			SP	Poorly-graded sands, gravelly sands, little or no fines.	Predominantly one size or a range of sizes with some intermediate sizes missing.		
		Sands with Fines (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures.	Nonplastic fines or fines with low plasticity. (for identification procedures see ML below)		
			SC	Clayey sands, sand-clay mixtures.	Plastic fines (for identification procedures see CL below).		
Fine-grained Soils More than half of material is smaller than 75 µm (No. 200) sieve size					Identification Procedures on Fraction Smaller than 425 µm (No. 40) Sieve Size		
					Dry Strength (Crushing characteristics)	Dilatancy (Reaction to shaking)	Toughness (Consistency near PL)
	Silts and Clays Liquid limit less than 50		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.	None to slight	Quick to slow	None
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	Medium to high	None to very slow	Medium
			OL	Organic silts and organic silty clays of low plasticity.	Slight to medium	Slow	Slight
	Silts and Clays Liquid limit greater than 50		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	Slight to medium	Slow to none	Slight to medium
			CH	Inorganic clays of high plasticity, fat clays.	High to very high	None	High
			OH	Organic clays of medium to high plasticity, organic silts.	Medium to high	None to very slow	Slight to medium
Highly Organic Soils			Pt	Peat and other highly organic soils.	Readily identified by color, odor, spongy feel and frequently by fibrous texture.		

(1) Boundary classifications: Soils possessing characteristics of two groups are designated by combinations of group symbols. For example GW-GC well-graded gravel sand mixture with clay binder. (2) All sieve sizes on this chart are U.S. standard.

U.S.A. Standard Series Sieve Designations	
Standard	Alternate
75 mm	3 in.
12.5 mm	½ in.
6.3 mm	¼ in.
4.75 mm	No. 4
2.00 mm	No. 10
425 µm	No. 40
75 µm	No. 200

FIELD IDENTIFICATION PROCEDURES FOR FINE-GRAINED SOILS OR FRACTIONS

These procedures are to be performed on the minus 425 µm (No. 40) sieve size particles, approximately 0.4mm (1/64 in.). For field classification purposes, screening is not intended, simply remove by hand the coarse particles that interfere with the tests.

Dilatancy (Reaction to shaking)

After removing particles larger than 425 µm (No. 40) sieve size, prepare a pat of moist soil with a volume of about 8 ml (one-half cubic inch). Add enough water if necessary to make soil soft but not sticky. Place the pat in the open palm of one hand and shake horizontally, striking vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pat which changes to a livery consistency and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear from the surface, the pat stiffens, and finally it cracks or crumbles. The rapidity of appearance of water during shaking and of its disappearance during squeezing assist in identifying the character of the fines in a soil.

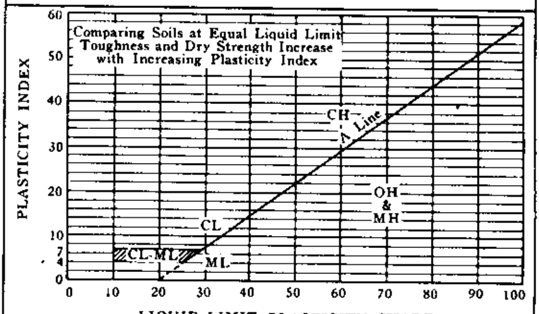
Very fine clean sands give the quickest and most distinct reaction whereas a plastic clay has no reaction. Inorganic silts, such as a typical rock flour, show a moderately quick reaction.

Dry Strength (Crushing characteristics)

After removing particles larger than 425 µm (No. 40) sieve size, mold a pat of soil to the consistency of putty, adding water if necessary. Allow the pat to dry

classification is checked and refined by employing common items of laboratory equipment to perform simple and routine tests, such as gradation, liquid limit, and plastic limit. Gradation is determined by sieve analysis. Plasticity characteristics are evaluated using the liquid limit and plastic limit tests on the soil fraction finer than the 425 µm (No. 40) sieve.

Table 10.4 Unified Soil Classification (Including Identification and Description) (Continued)

Information Required for Describing Soils	Laboratory Classification Criteria		
6	7		
For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics.	Use grain-size curve in identifying the fractions as given under field identification. Determine percentages of gravel and sand from grain-size curve. Depending on percentage of fines (fraction smaller than 75 μm No. 200 sieve size) coarse-grained soils are classified as follows: Less than 5% GW, GP, SW, SP, More than 12% GM, GC, SM, SC. 5% to 12% Borderline cases requiring use of dual symbols.	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 4 $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between one and 3	
		Not meeting all gradation requirements for GW	
Give typical name; indicate approximate percentages of sand and gravel, max. size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbol in parentheses.		Atterberg limits below "A" line or PI less than 4	Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols.
		Atterberg limits above "A" line with PI greater than 7	
Example: Silty sand gravelly; about 20% hard, angular gravel particles 13 mm (½ in.) maximum size; rounded and sub-angular sand grains coarse to fine; about 15% nonplastic fines with low dry strength; well compacted and moist in place; alluvial sand; (SM).		$C_u = \frac{D_{60}}{D_{10}}$ Greater than 6 $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between one and 3	
		Not meeting all gradation requirements for SW	
		Atterberg limits below "A" line or PI less than 4	Limits plotting in hatched zone with PI between 4 and 7 are borderline cases requiring use of dual symbols.
		Atterberg limits above "A" line with PI greater than 7	
Give typical name, indicate degree and character of plasticity, amount and maximum size of coarse grains, color in wet condition, odor if any, local or geologic name, and other pertinent descriptive information; and symbol in parentheses. For undisturbed soils add information on structure, stratification, consistency in undisturbed and remolded states, moisture and drainage conditions. Example: Clayey silt, brown, slightly plastic, small percentage of fine sand, numerous vertical root holes, firm and dry in place, loess, (ML).		 LIQUID LIMIT PLASTICITY CHART For laboratory classification of fine-grained soils	

completely by oven, sun, or air drying, and then test its strength by breaking and crumbling between the fingers. This strength is a measure of the character and quantity of the colloidal fraction contained in the soil. The dry strength increasing plasticity.

The dry strength is a characteristic for clays of the CH group. A typical inorganic silt possesses only very slight dry strength. Silty fine sands and silts have about the same slight dry strength, but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty whereas a typical silt has the smooth feel of flour.

Toughness (Consistency near plastic limit)

After removing particles larger than the 425 μm (No. 40) sieve size, a specimen of soil about 13 mm (one-half inch) cube in size is molded to the consistency of putty. If too dry, water must be added and if sticky, the specimen should spread out in a thin layer and allowed to lose some moisture by evaporation. Then the specimens is rolled out by hand on a smooth surface or between the palms into a thread about 3 mm (one-eighth inch) in diameter. The thread is then folded and rerolled repeatedly. During this manipulation the moisture content is gradually reduced and the specimen stiffens, finally loses its plasticity, and crumbles when the plastic limit is reached.

After the thread crumbles, the pieces should be lumped together and a slight kneading action continued until the lump crumbles.

The tougher the thread near the plastic limit and the stiffer the lump when it finally crumbles, the more potent is the collidal clay fraction in the soil. Weakness of the thread at the plastic limit and quick loss of coherence of the lump below the plastic limit indicate either inorganic clay of low plasticity, or materials such as kaolin-type clays and organic clays which occur below the A-line.

Highly organic clays have a very weak and spongy feel at the plastic limit.

The criteria for classifying soils are given in column 7 of Table 10.4. The plasticity chart (Figure 10.2) is used to classify the fine-grained soils and the fine-grained portion of coarse-grained soils.

Test procedures for classifying soils are:

Standard Tests	**AASHTO**	**ASTM**
Amount of Material Finer than 75 μm (No. 200)	T 11	D 1140*
Sieve Analysis of Fine and Coarse Aggregate	T 27	C 136*
Specific Gravity of Soil	T 100	D 854*
Particle Size Analysis of Soils	T 88	D 422*
Liquid Limit of Soils	T 89	D 4318*
Plastic Limit and Plasticity Index of Soils	T 90	D 4318*

*These ASTM test procedures differ in some respects from their AASHTO counterparts.

10.2 Soil-Subgrade Strength Evaluation Methods

Introduction

Need for Strength Evaluation

Thickness requirements for asphalt pavements depend to a large extent on the strength of the finished subgrade. The concept of testing soils at their anticipated moisture and density should be the governing criterion for thickness design.

Stability or bearing capacity is closely related to density and moisture content. Variable strength can be expected, depending on the degree of compaction and moisture content after the pavement has been completed. If experience has indicated that a certain soil cannot be compacted economically to high density, it would be unwise to perform design tests with the soil at a high density value.

Design strength should be based on values obtained from soil samples in their worst anticipated condition. To obtain such values generally involves testing a sample that has been soaked in water.

Evaluation Methods

Design methods employed by different agencies vary widely, and their approach to evaluating the strength of the subgrade for design purposes also varies. Evaluation methods discussed here are probably the most widely used methods for asphalt pavement design for airfields and highways. They are:

- Plate Bearing test
- California Bearing Ratio (CBR) method
- Resistance Value Method (R-Value)
- Resilient Modulus (M_r) method.

During 1945 and 1946 the Canadian Department of Transport undertook a comprehensive investigation of runways at a number of principal airports in Canada. This work was reported by Norman W. McLeod. One of the objectives was to obtain test data that could be employed in designing flexible pavements on the basis of repetitive plate bearing tests performed on the subgrade or pavement layers already in place.

The California Bearing Ratio test (CBR), with its numerous variations, is probably the most widely used method of designing asphalt pavement structures. This method was first developed by the California Division of Highways around 1930. It has since been adopted and modified by numerous states, the U.S. Corps of Engineers, and many countries, although it is no longer used by the California Department of Transportation.

A method of evaluating treated and untreated materials for bases, subbases, and subgrades for pavement thickness design was developed by

F.N. Hveem and R.M. Carmany of the California Division of Highways. It is known as the Resistance Value (R-Value) method. This method is currently used by the highway departments of several other states in addition to California.

The Resilient Modulus (M_r) method was developed by the Asphalt Institute based on a test originated at the University of California by H.B Seed, C.K. Chan, and C.E. Lee. It is used to determine the resilient character of a soil specimen for conditions that represent stress in pavements subjected to moving wheel loads.

Plate Bearing Test

General

Plate bearing tests are used by a number of organizations for the design and evaluation of asphalt pavement structures. Test procedures employed may vary somewhat between agencies, but the method discussed here is from *Repetitive Static Plate Load Tests of Soils and Flexible Pavement Components for Use in Evaluation and Design of Airport and Highway Pavements* (ASTM D 1195).

Like all other soil strength tests, a plate bearing test does not provide a representative measurement of subgrade strength, unless it is made when the soil is in the same condition as that expected after the environmental influences of moisture, density, frost, drainage, and traffic have been present. When the strength of the subgrade soil for a new highway or airport pavement is required, this may be achieved by one of the following methods:

1. Perform the test on the subgrade beneath an existing asphalt pavement where the same subgrade soil occurs, and where the pavement has been in place long enough for the subgrade to have reached an equilibrium with its environment.

2. Perform the test on a specially constructed test section of the soil, where an adequate depth has been processed to duplicate the conditions expected after the pavement has been in place for a period of time.

Test Procedure

Figure 10.3 shows an arrangement of the equipment used in performing the repetitive load test. Criteria for the thickness design requirements for highways and airports are based on:

	Highways	**Runways**
Bearing plate diameter	300 mm (12 in.)	760 mm (30 in.)
Deflection	5 mm (0.2 in.)	13 mm (0.5 in.)
Load repetitions	10	10

The loading device can be a truck, trailer, a combination of both a tractor and trailer, an anchored frame, or other structure. It is loaded with sufficient weight to produce the desired reaction on the surface under test. The supporting points (wheels in the case of a truck or trailer) should be at least 2.5 m (8 ft) from the circumference of the largest diameter plate being used.

As a first step in the loading procedure, the bearing assembly is seated by a quick application and release of a load sufficient to produce a deflection 0.25 to 0.5 mm (0.01 to 0.02 in.). One-half the seating load is reapplied to the plate and the deflection gauges are set at zero.

A load giving a deflection of about 1 mm (0.04 in.) is applied and maintained until the increase in deflection is 0.03 mm (0.001 in.) or less per minute for 3 consecutive minutes. The load is then completely released and the recovery is observed until the rate of rebound becomes 0.03 mm (0.001 in.) or less per minute. This procedure of loading and unloading with the same load is repeated until ten complete applications have been made.

At the conclusion of that series of load applications and releases, the load is increased to an amount required to produce a deflection of about 5 mm (0.2 in.); the same procedure is repeated with the increased load. The load is then increased to an amount required to produce a deflection of about 10 mm (0.4 in.), and again the same procedure is employed with the new load.

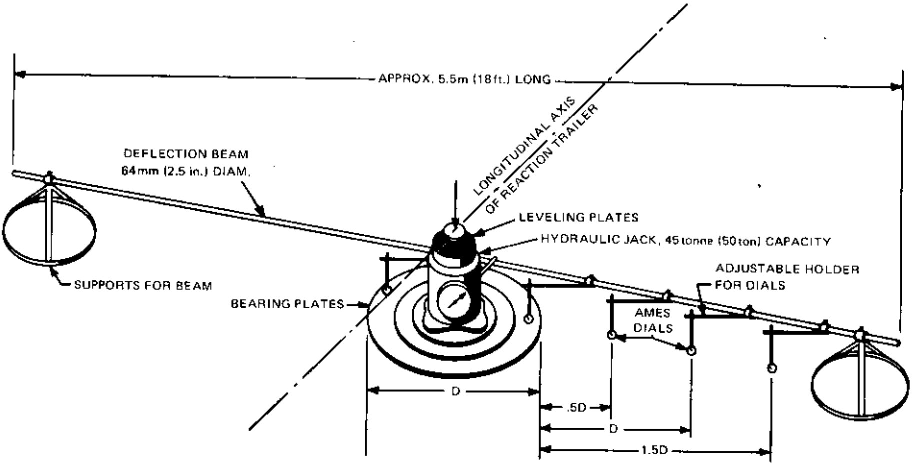

APPROX. 5.5m (18ft.) LONG
LONGITUDINAL AXIS OF REACTION TRAILER
DEFLECTION BEAM 64mm (2.5 in.) DIAM.
LEVELING PLATES
HYDRAULIC JACK, 45 tonne (50 ton) CAPACITY
ADJUSTABLE HOLDER FOR DIALS
SUPPORTS FOR BEAM
BEARING PLATES
AMES DIALS
D
.5D
D
1.5D

Figure 10.3 Arrangement of Equipment for Plate Bearing Test

Calculating and Plotting Data

For each repetition of each load, the terminal deflection at which the rate of deflection is 0.03 mm (0.001 in.) per minute is determined. Corrections are made as necessary for deflections and load values. The deflections vs. load repetitions may then be plotted (Figure 10.4) to show influence on deflection of repetitions of loading. A total load versus average deflection for ten repetitions of load applications (Figure 10.5) is plotted and the plate bearing value determined from a curved line connecting the points.

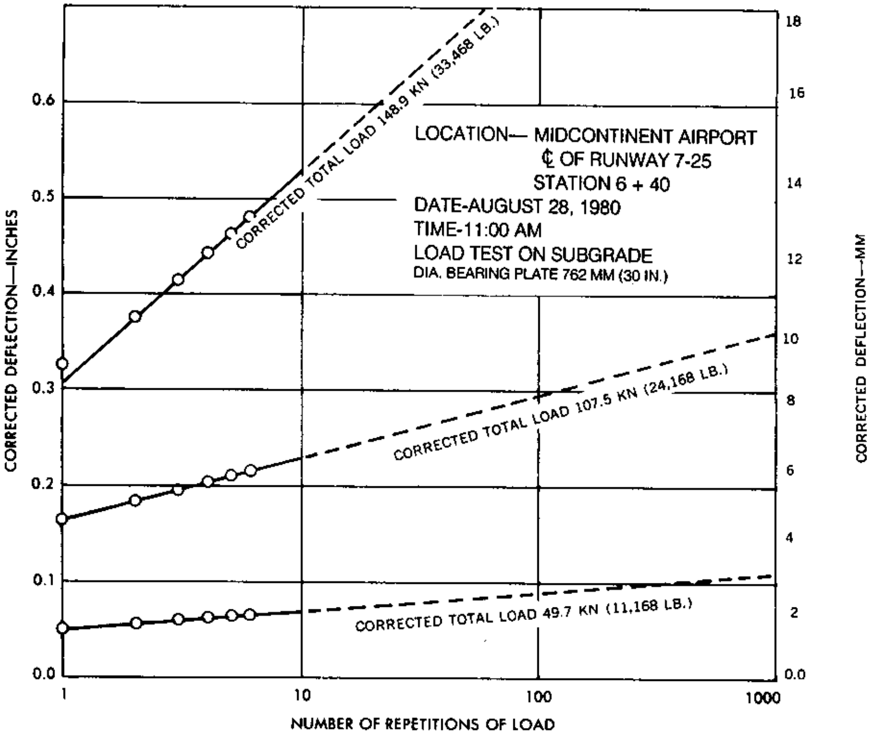


Figure 10.4 Example Showing Influence on Deflection of Repetitions of Loading for Each of Three Magnitudes of Load.

California Bearing Ratio (CBR) Method

General

The California Bearing Ratio (CBR) Method is probably the most widely used method of designing asphalt pavement structures. This method was developed by the California Division of Highways around 1930 and has since been adopted and modified by numerous states, the U.S. Corps of Engineers, and many countries of the world. The Corps of Engineers adopted this method during the 1940s. Their test procedure was most generally used until 1961 when the American Society for Testing and Materials adopted the method as ASTM Designation D 1883, *Bearing Ratio of Laboratory-Compacted Soils.* The ASTM Procedure differs in some respects from the Corps' procedure and from the American Association of State Highway and Transportation Officials' procedure, AASHTO Designation T 193. The ASTM procedure is the easiest to use, and is the version described in this section.

The CBR is a comparative measure of the shearing resistance of soil. The test consists of measuring the load required to cause a piston of standard size to penetrate a soil specimen at a specified rate. This load is divided by the

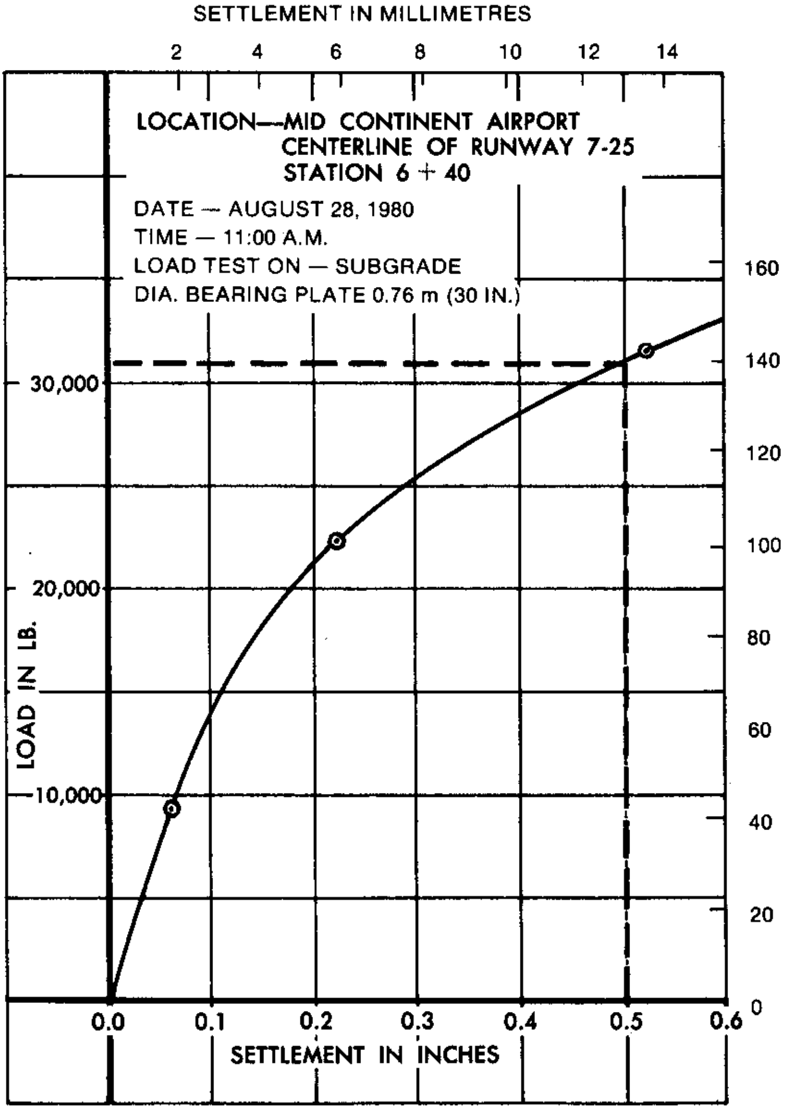


Figure 10.5 **Corrected Total Load Versus Corrected Deflection for 10 Repetitions of Load.**

load required to force the piston to the same depth in a standard sample of crushed stone. The result, multiplied by 100, is the value of the California Bearing Ratio (CBR). Usually depths of 2.5 or 5 mm (0.1 to 0.2 in.) are used, but depths of 7.5, 10 and 12.5 mm (0.3, 0.4 and 0.5 in.) may be used if desired. Penetration loads for the crushed stone have been standardized. This test method is intended to provide the relative bearing value, or CBR, of base, subbase, and subgrade materials. Procedures are given for laboratory-compacted swelling, nonswelling, and granular materials. These tests are usually performed to obtain information that will be used for design purposes.

The CBR value for a soil will depend upon its density, molding moisture content, and moisture content after soaking. Since the product of laboratory compaction should closely represent the results of field compaction, the first two of these variables must be carefully controlled during the preparation of laboratory samples for testing. Unless it can be ascertained that the soil being tested will not accumulate moisture and be affected by it in the field after construction, the CBR tests should be performed on soaked samples.

There are certain routine soil tests that should be performed prior to conducting the CBR test. These tests are as follows:

	ASTM	AASHTO
Sieve Analysis of Fine and Coarse Aggregate	C 136	T 27
Liquid Limit of Soils	D 4318	T 89
Plastic Limit and Plasticity Index of Soils	D 4318	T 90
Particle Size Analysis of Soils (only for classification purposes)	D 422	T 88
Moisture-Density Relations of Soils Using 2.5 kg (5.5 lb) Rammer	D 698	T 99
Moisture-Density Relations of Soils Using 4.5 kg (10 lb) Rammer	D 1577	T 180

Preparing Test Specimens

If the sample is to be soaked, a representative sample of the material is taken for the determination of moisture at the beginning of compaction, and another sample of the remaining material after compaction. The material is weighed immediately and dried in an oven at 110±5° C (230±9° F) for at least 12 hours, or to constant weight. Each moisture content sample should weigh not less than 100g for fine-grained soils nor less than 500g for granular soils. If the sample is not to be soaked, a representative sample of material is taken from one of the cut faces after penetration, to determine the moisture content.

The apparatus for the California Bearing Ratio test is shown in Figure 10.6. The mold, with extension collar attached, is clamped to the base plate. The spacer disk is inserted over the base plate, and coarse filter paper is placed on top of the spacer disk. The soil sample is compacted at maximum density and optimum moisture content as determined by ASTM D 698 or D 1557.

The extension collar is removed and the compacted soil trimmed even with the top of the mold with a straightedge. The spacer disk is removed and the mold inverted. The trimmed edge of the soil is placed on the perforated base plate which has been covered with filter paper. The adjustable stem and plate assembly is placed on top of the compacted soil specimen in the mold.

Surcharge weights within 2.27 kg (5 lb) of the weight of the base material and pavement are added. In no case, however, should the surcharge be less than 4.54 kg (10 lb). The sample is immersed in water and initial measurements taken for swell. After 96 hours, or a shorter period for soils that readily take up moisture, final swell measurements are made to determine swell as a percentage of initial specimen height. The sample is removed from the water and the surcharge weights and perforated plate removed. The sample is allowed to drain for 15 minutes after which time it is weighed and prepared for the penetration test.

Penetration Testing

The penetration test is conducted with surcharge weights in place, equal to those used during the soaking period. The mold assembly is placed in a loading frame or hydraulic press. The piston is first seated with an initial 44 N (10 lb) load, and all dials are set at zero. A load is then applied at the uniform rate of 1.27 mm (0.05 in.) penetration per minute. The total load readings are taken at the following penetrations:

0.6 mm (0.025 in.)
1.3 mm (0.050 in.)
1.9 mm (0.075 in.)
2.5 mm (0.100 in.)
3.2 mm (0.125 in.)
3.8 mm (0.150 in.)
4.4 mm (0.175 in.)
5.1 mm (0.200 in.)
6.4 mm (0.250 in.)
7.6 mm (0.300 in.)
10.1 mm (0.400 in.)
12.7 mm (0.500 in.)

Finally the load is released, the mold assembly disassembled, and the moisture-content is determined for the top 25 mm (1 in.), or the entire depth of the sample.

Calculating and Plotting Data

After the test has been completed, the penetration load in megapascals (psi) is calculated and the load-penetration curve plotted on cross-section paper. In order to obtain true penetration loads from the test data, the zero point of the curve is adjusted to correct for surface irregularities and the initial concave upward shape of the curve if it is present. If the curve is uniform as in example No. 1 of Figure 10.7, the CBR value is calculated from the recorded loads. For surface irregularities as in example No. 3 of Figure 10.7 extend the straight line portion of the curve to the base to obtain a corrected origin, or zero. If the curve has a reverse bend, or concave upward shape, as in example No. 2, draw a line tangent to the steepest point of the curve (point A), and extend the line to the base to obtain a corrected origin or zero point (point B). Then read the corrected load values for 2.5 mm (0.1 in.) penetration (point C) and 5 mm (0.2 in.) penetration (point D).

Penetration		Standard Load		Standard* Load	
mm	(in.)	N	(lb)	MPa	(psi)
2.5	(0.1)	13345	(3000)	6.89	(1000)
5.0	(0.2)	20017	(4500)	10.34	(1500)
7.5	(0.3)	25355	(5700)	13.10	(1900)
10.0	(0.4)	30693	(6900)	15.86	(2300)
12.5	(0.5)	34696	(7800)	17.93	(2600)

*Plunger cross-section area = 1935.5 mm^2 (3 in.^{2})

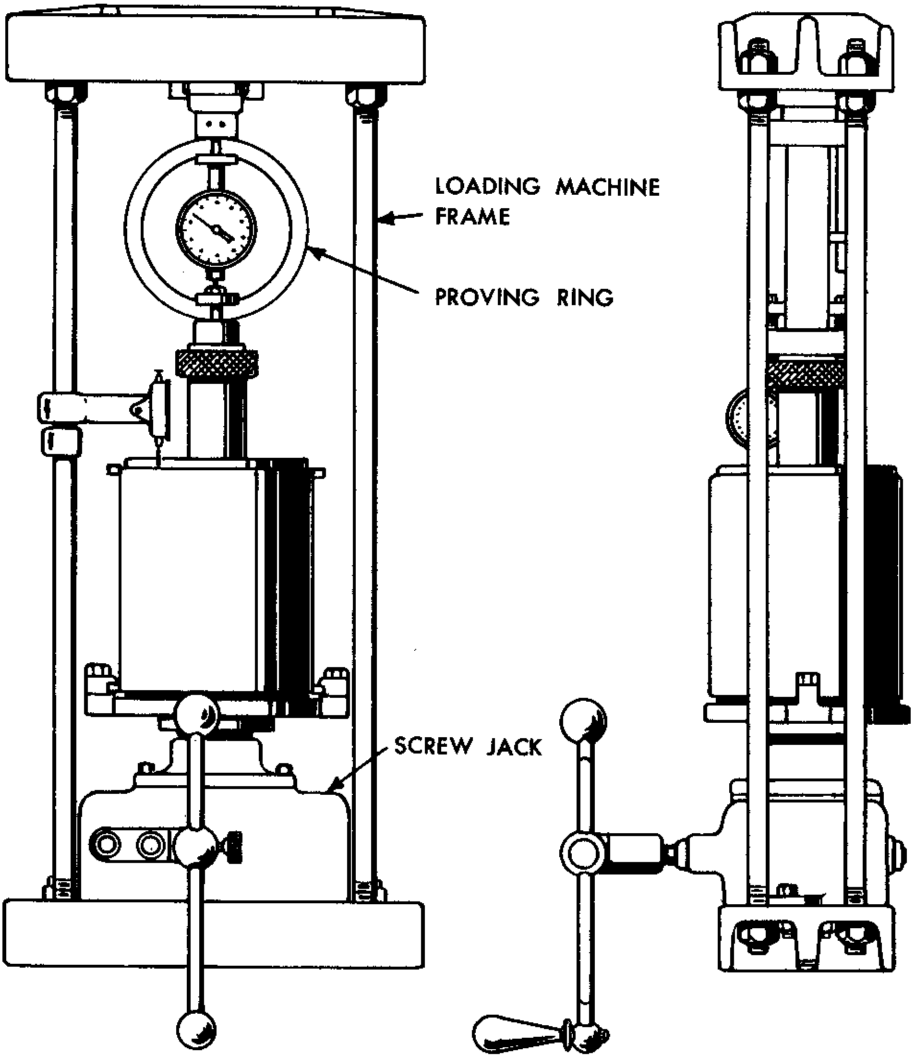


Figure 10.6 CBR Test Assembly.

The CBR value is defined as a ratio comparing the bearing of a material with the bearing of a well-graded crushed stone. The penetration loads for crushed stone are presented on the previous page.

The CBR is determined from the corrected load values at 2.5 and 5 mm (0.1 and 0.2 in.) penetrations by dividing the loads at 2.5 and 5 mm (0.1 and 0.2 in.) by the standard loads of 6.89 and 10.34 MPa (1000 and 1500 psi), respectively. Each ratio is multiplied by 100 to obtain the CBR in percent. The CBR is usually selected at 2.5 mm (0.1 in.) penetration. If the CBR at 5 mm (0.2 in.) penetration is greater, the test should be rerun. If check tests give similar results, the CBR at 5 mm (0.2 in.) penetration should be used. Figure 10.8 is a suggested form for recording CBR data.

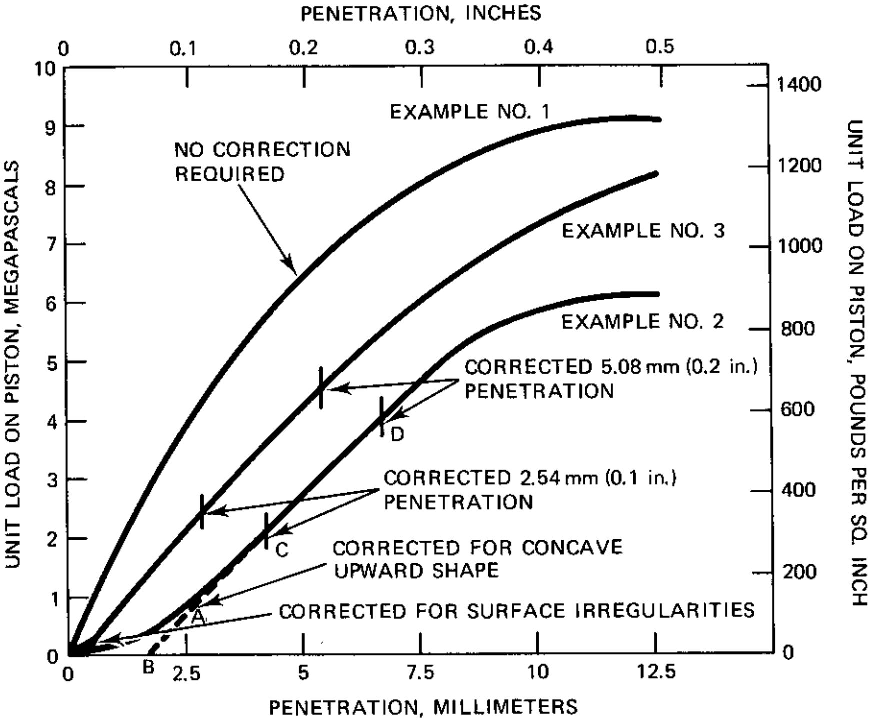


Figure 10.7 Correction of Load-Penetration Curves.

Resistance Value (R-Value) Method

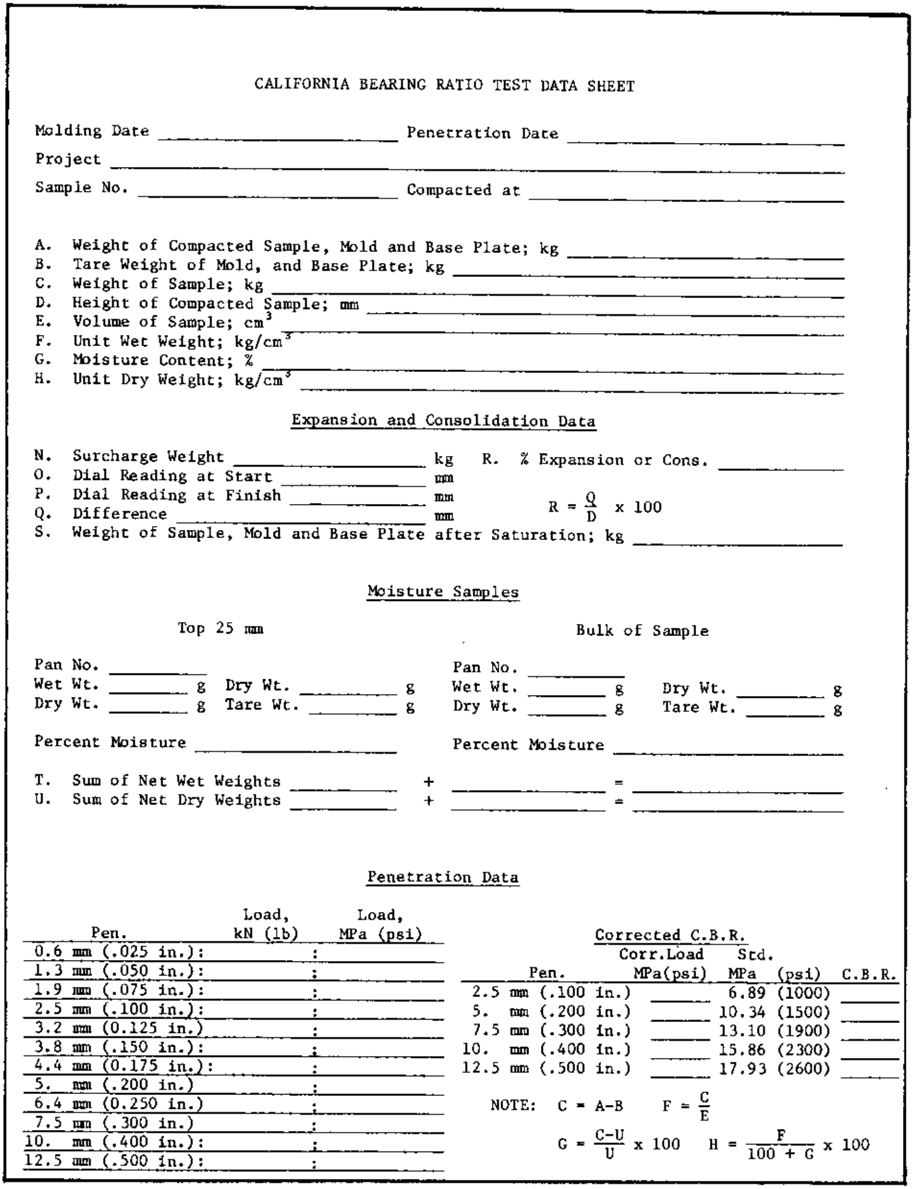
General

The R-value method of design is based on two separate measurements: the R-value (or resistance value) and the expansion pressure test.

The cover thickness or structural section required to prevent plastic deformation of soil under imposed wheel loads is determined by the R-value. An expansion pressure test is used to determine the thickness or weight of cover required to maintain soil compaction. The design R-value is determined from the moisture content and density at which these two thicknesses are equal. With granular nonexpansive soils, the design R-value is determined for a density considered to be equivalent to that obtained by normal construction compaction. This density value is obtained from the exudation pressure data.

The test method for determining the R-value consists of:

1. Preparation of samples,
2. Compaction of test specimens,
3. Determination of exudation pressure of test specimens,
4. Determination of expansion pressures of test specimens, and
5. Determination of R-value.



CALIFORNIA BEARING RATIO TEST DATA SHEET

Molding Date ________ Penetration Date ________
Project ________
Sample No. ________ Compacted at ________

A. Weight of Compacted Sample, Mold and Base Plate; kg ________
B. Tare Weight of Mold, and Base Plate; kg ________
C. Weight of Sample; kg ________
D. Height of Compacted Sample; mm ________
E. Volume of Sample; cm^3 ________
F. Unit Wet Weight; kg/cm^3 ________
G. Moisture Content; % ________
H. Unit Dry Weight; kg/cm^3 ________

Expansion and Consolidation Data

N. Surcharge Weight ________ kg
O. Dial Reading at Start ________ mm
P. Dial Reading at Finish ________ mm
Q. Difference ________ mm
R. % Expansion or Cons. ________

$$R = \frac{Q}{D} \times 100$$

S. Weight of Sample, Mold and Base Plate after Saturation; kg ________

Moisture Samples

Top 25 mm

Pan No. ________
Wet Wt. ________ g Dry Wt. ________ g
Dry Wt. ________ g Tare Wt. ________ g
Percent Moisture ________

Bulk of Sample

Pan No. ________
Wet Wt. ________ g Dry Wt. ________ g
Dry Wt. ________ g Tare Wt. ________ g
Percent Moisture ________

T. Sum of Net Wet Weights ________ + ________ = ________
U. Sum of Net Dry Weights ________ + ________ = ________

Penetration Data

Pen.	Load, kN (lb)	Load, MPa (psi)
0.6 mm (.025 in.):	:	
1.3 mm (.050 in.):	:	
1.9 mm (.075 in.):	:	
2.5 mm (.100 in.):	:	
3.2 mm (0.125 in.)	:	
3.8 mm (.150 in.):	:	
4.4 mm (0.175 in.):	:	
5. mm (.200 in.)	:	
6.4 mm (0.250 in.)	:	
7.5 mm (.300 in.)	:	
10. mm (.400 in.):	:	
12.5 mm (.500 in.):	:	

Corrected C.B.R.

Pen.	Corr.Load MPa(psi)	Std. MPa (psi)	C.B.R.
2.5 mm (.100 in.)	____	6.89 (1000)	____
5. mm (.200 in.)	____	10.34 (1500)	____
7.5 mm (.300 in.)	____	13.10 (1900)	____
10. mm (.400 in.)	____	15.86 (2300)	____
12.5 mm (.500 in.)	____	17.93 (2600)	____

NOTE: $C = A - B$ $\quad F = \frac{C}{E}$

$$G = \frac{C-U}{U} \times 100 \qquad H = \frac{F}{100 + G} \times 100$$

Figure 10.8 Sample Form for Recording CBR Test Information.

Preparation of Samples

The R-value test is made on a relatively small sample, and it is necessary to remove all large-size material. If 75 percent or more of the material passes the 19.0 mm (¾ in.) sieve, the material retained on that sieve is removed. If less than 75 percent of the material passes the 19.0 mm (¾ in.) sieve, then all material retained on the 25.0 mm (1 in.) sieve is removed. The soil is then separated over the 4.75 mm (No. 4) sieve but recombined in the proper proportions with the coarse sizes to obtain 1200-g samples plus an extra 200 g for moisture content determinations.

In each R-value test, four test briquettes at different moisture contents are prepared. Each briquette has ½ to ²/₃ the amount of water necessary to saturate the sample or exude moisture between 690 to 5500 kPa (100 and 800 psi). After mixing, loose material is allowed to cure overnight in a covered container. Then more water is added and mixed until the estimated saturation moisture is attained. The first briquette is used as a pilot specimen to obtain specimen heights of 63.5 ±2.5 mm (2.5±0.1 in.) and to have exudation pressures both above and below 2070 kPa (300 psi).

Compaction of Test Specimens

Specimens are compacted by a kneading compactor (Figure 10.9), which is the same as used for the Hveem method of asphalt mix design. The compaction procedure provides a kneading action that is intended to give soil particles the same orientation and contact pressure obtained during field compaction.

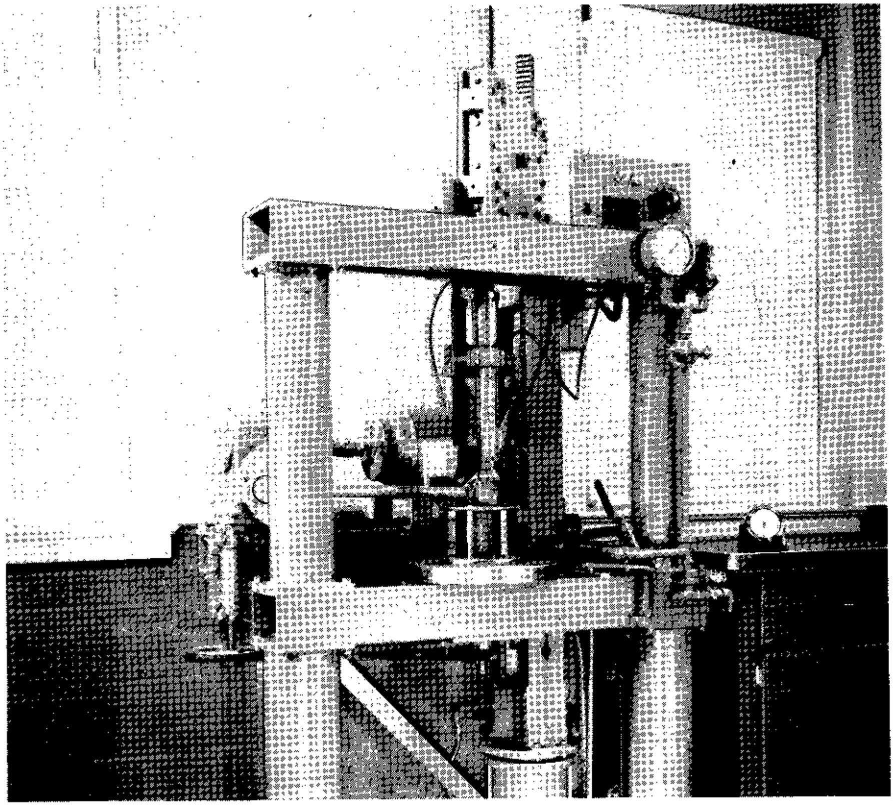


Figure 10.9 Mechanical Compactor.

The prepared soil sample is placed in the compactor feeding trough (Figure 10.10) with the material distributed evenly along the full length. A 101.6 mm (4 in.) diameter by 127 mm (5 in.) high mold is fitted into a special holder for placing in the compactor. Material is deposited in the mold by a special procedure while the kneading compactor is operating and the foot pressure is set at 1720 kPa (250 psi).

When all material has been placed in the mold and 10 extra blows have been added to level and seat the material, the sample is compacted by using 100 blows with a foot pressure of 2400 kPa (350 psi).

Some clays and clean sands require lower compaction pressures. It may be necessary, if the surface is uneven, to smooth and level the tamped surface.

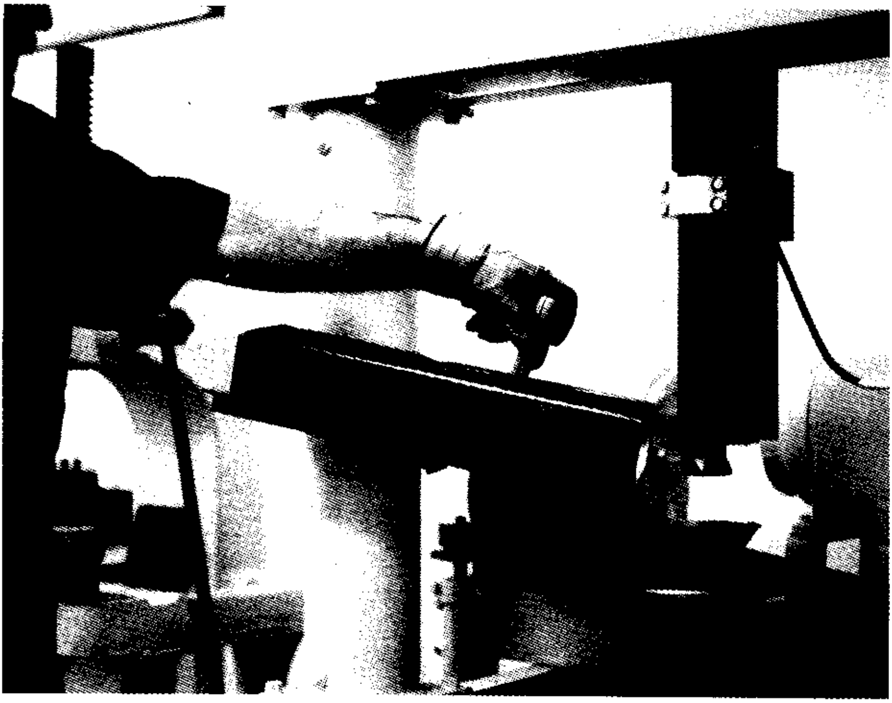


Figure 10.10 Depositing Soil Into Mold From Compactor Feeding Trough.

Determination of Exudation Pressure

The exudation pressure test is used to determine the compressive stress necessary to exude water from the compacted specimen. Results from tests on several specimens are used to determine the R-value at a standard pressure of 2070 kPa (300 psi).

In this test a bronze disk is placed on top of the tamped specimen in the mold, and a piece of filter paper is placed on the disk. The assembly is then inverted so that the filter paper is on the bottom. Before the assembly is positioned in the testing machine, it is placed on the contact plate of the moisture exudation indicator (Figure 10.11).

A ram is used to force the sample down in the mold to the contact plate of the exudation indicator. The testing machine then applies a load to the ram at the rate of 8.9 kN (2000 lb) per minute until water exuding from the sample on the indicator plate lights five of the six outer lights of the indicator. Total load on the sample at that moment is converted to kPa (psi) and recorded as exudation pressure. Specimens having exudation pressures of less than 690 kPa (100 psi) or more than 5500 kPa (800 psi) are usually discarded.

Upon completion of the test, the specimen is left in a covered mold for at least ½ hour before the next part of the test procedure is begun.

Determination of Expansion Pressure

The expansion pressure test is used in determining the expansion pressure a soil develops under water. This test is performed in such a way that a large change in sample density does not occur during testing. Pavement thicknesses required to withstand expansion pressures can be determined and compared with those determined by the R-Value test.

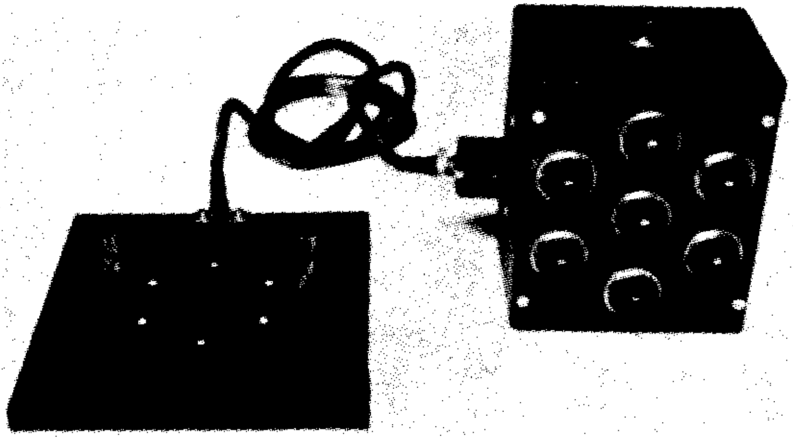


Figure 10.11 Moisture Exudation Indicating Device.

The specimen is allowed to stand for at least ½ hour after the completion of the exudation test. A perforated brass plate with stem is placed on the face of the compacted specimen in the mold. The mold is then positioned in the calibrated expansion device (Figure 10.12), and 200 ml of water is poured on the specimen in the mold. Expansion pressure is allowed to develop from 16 to 24 hours, after which the deflection of the calibrated spring steel bar is read and the expansion pressure determined.

Determination of R-Value

The R-value test provides a rapid means for determining a measure of the stability or resistance to plastic deformation of compacted materials by the Hveem stabilometer (Figure 10.13). It is the same device used for the Hveem method of asphalt mix design.

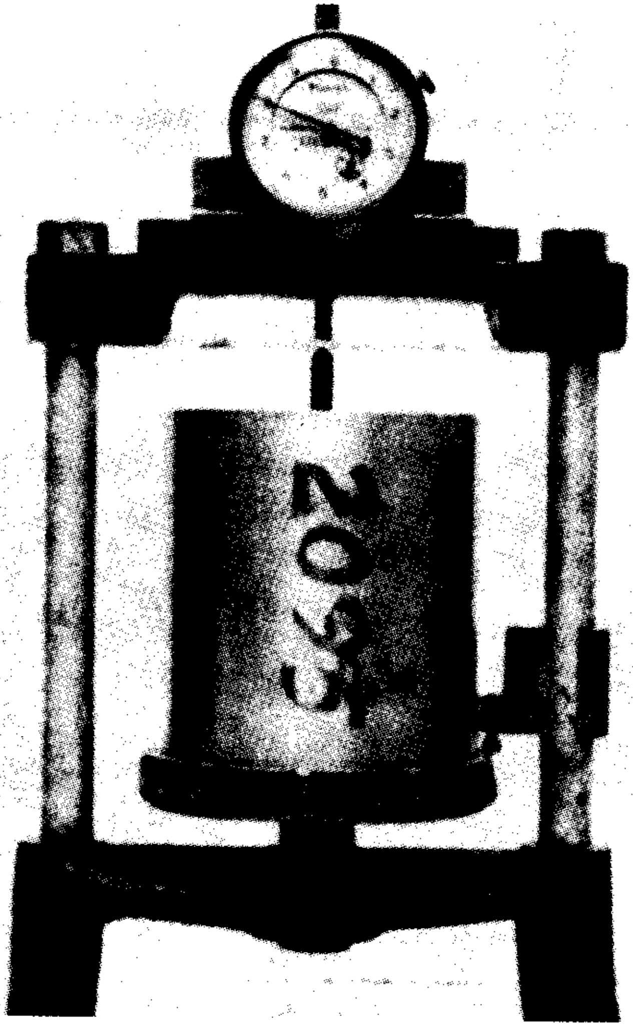


Figure 10.12 Expansion Pressure Assembly.

The specimen that was tested for expansion pressure is forced from the mold into an adjusted stabilometer. The stabilometer is then positioned in the testing machine. The head is lowered until it contacts the specimen but applies no load. A lateral pressure of 35 kPa (5 psi) is then applied with the

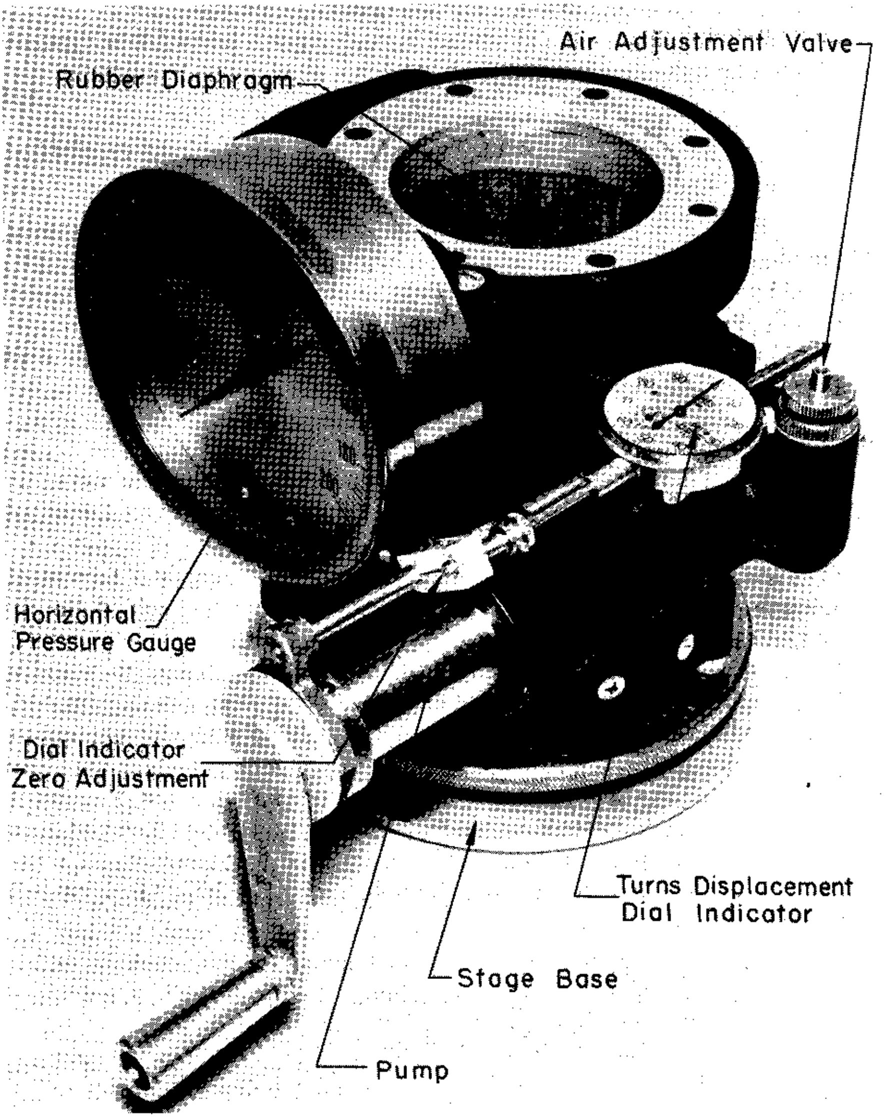

Air Adjustment Valve
Rubber Diaphragm
Horizontal Pressure Gauge
Dial Indicator Zero Adjustment
Turns Displacement Dial Indicator
Stage Base
Pump

Figure 10.13 Hveem Stabilometer.

stabilometer pump. Next the specimen is loaded at the rate of 1.3 mm (0.05 in.) per minute and lateral pressure readings are obtained at 550 and 1100 kPa (80 and 160 psi) vertical pressure.

Vertical loading must be stopped at 8.9 kN (2000 lb) and reduced immediately to 4.45 kN (1000 lb). The lateral pressure is reduced to 35 kPa (5 psi). The turns displacement dial indicator is set at zero and the pump handle is turned at two turns per second until the stabilometer gauge reads 690 kPa

(100 psi). The number of turns are recorded, and the stabilometer R-value is then calculated from:

$$R = 100 - \frac{100}{\frac{2.5}{D}\left(\frac{P_v}{P_h} - 1\right) + 1} \qquad (1)$$

where

D = turns displacement reading,
P_v = 1100 kPa (160 psi) vertical pressure,
P_h = horizontal pressure [stabilometer gauge reading 1100 kPa (160 psi) vertical pressure].

Design R-Value

Thickness design of asphalt pavements will be discussed in detail in Chapter 11. The design R-value, however, cannot be determined without using a thickness design chart.

A thickness design chart is used to determine the required pavement thicknesses indicated by stabilometer—corresponding to each of the R-values classified.

Pavement thickness (from charts in Chapter 11) needed to provide enough weight of cover to equal the expansion pressures are determined. The required thicknesses by stabilometer vs. thicknesses by expansion pressure are plotted (Figure 10.14), and the thickness value at which the curve connecting the points crosses the 45 degree balance line is noted. Pavement thicknesses (by stabilometer R-values) are then plotted versus the exudation pressures for the three samples. The thickness is determined at the intersection of the curve connecting the points with the 2070 kPa (300 psi) line.

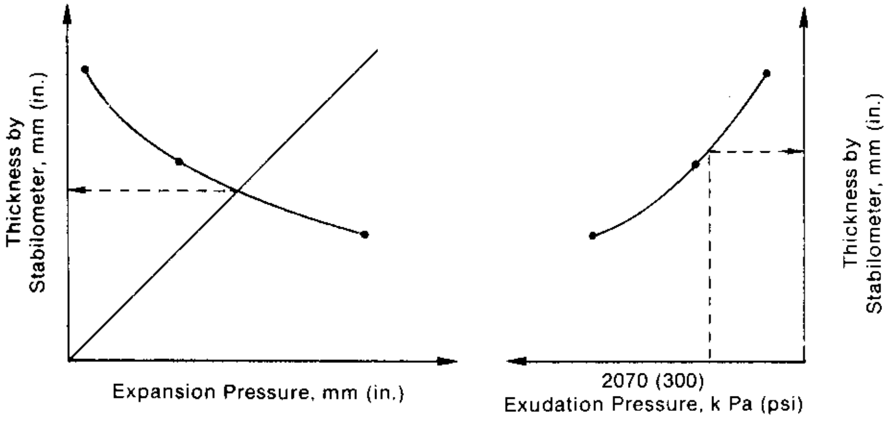


Figure 10.14 Plot of Laboratory Test Results and Required Pavement Thickness for Determining Design R-Value.

These two thicknesses are then compared. The greater value is used to re-enter the thickness design chart (Chapter 11). By working backward, the corresponding R-value to be used for design purposes is determined.

Resilient Modulus (M_r) Method

General

This method of test covers the determination of resilient modulus of untreated fine-grained soils for conditions that represent stress in pavements subjected to moving wheel loads. The results of the test describe the resilient character of a soil specimen when a repeated axial deviator stress of fixed magnitude, frequency, and load duration is applied while the specimen, in a triaxial chamber, is surrounded by pressure that acts as a static, all-around stress.

Equipment

A triaxial cell suitable for use in resilience testing of soils is shown in Figure 10.15. The equipment is similar to most standard cells, except that it is somewhat larger to facilitate the internally mounted load and deformation measuring equipment, and has additional outlets for the electrical leads from the measuring devices. (Triaxial cells with suitable externally mounted load and deformation measuring equipment also may be used.) For the type of equipment shown, air is used as the cell fluid.

The deformation measuring equipment consists of Linear Variable Differential Transformers (LVDTs) attached to the soil specimen by a pair of clamps. Two LVDTs are used for the measurement of axial deformation. The clamps and LVDTs are shown in position on a soil specimen in Figure 10.15. Preferably, load is measured by placing a load cell between the sample cap and the loading piston as shown in Figure 10.15. In addition to measuring devices, it is also necessary to maintain suitable signal excitation, conditioning, and recording equipment.

Loading

The external loading source may be any device capable of providing varying repeated loads in fixed cycles of load and release. These devices range from simple cam and switch control of static weights or air pistons to closed loop electro-hydraulic systems. A load duration of 0.1 second and load-release duration of from 0.4 to 3 seconds has been found to be satisfactory for most applications.

The confirming pressure and deviator stress used should approximate the expected in-place state of stresses. A confining pressure of 14 kPa (2 psi) and deviator stress of 41 kPa (6 psi) are reasonable values for most testing.

The test is started by applying 200 repetitions of a deviator stress of approximately 21 kPa (3 psi) followed successively by 200 repetitions each at 41 and 62 kPa (6 and 9 psi). The foregoing stress sequence constitutes sample

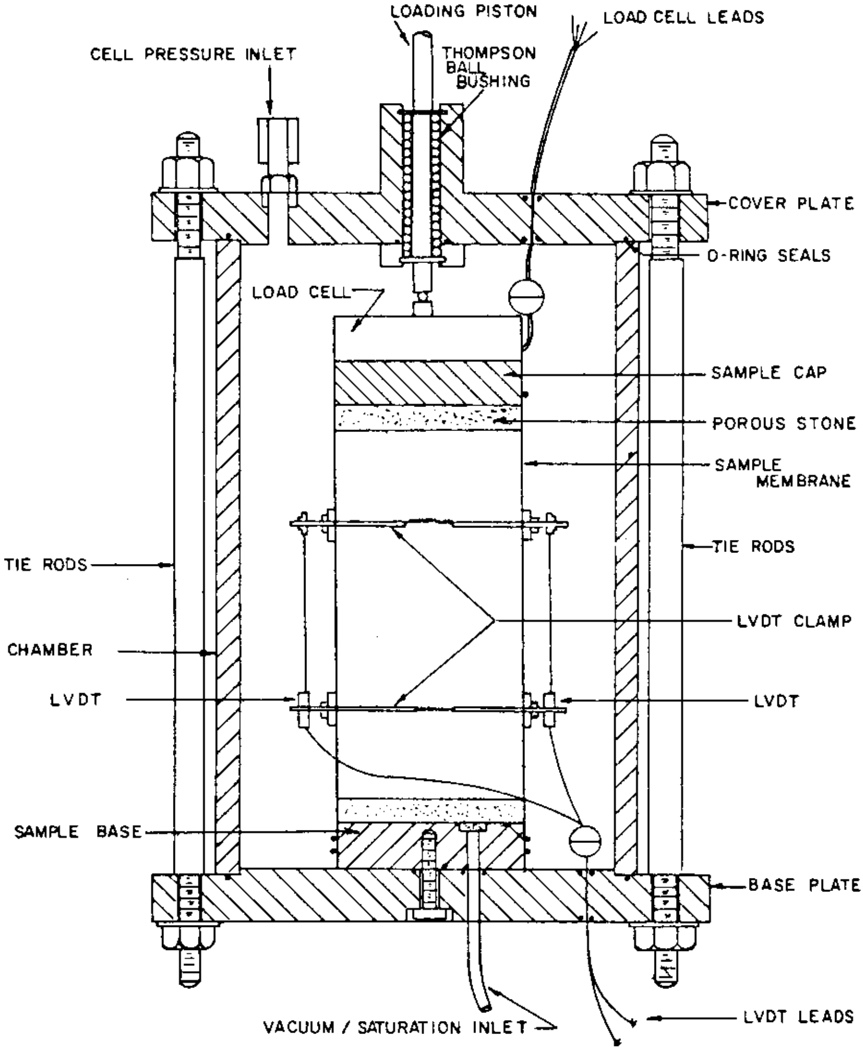


Figure 10.15 Apparatus for Resilient Modulus Testing of Soil.

conditioning, that is, elimination of the effects of the interval between compaction and loading and also the effects of initial load versus reloading. The deviator load is decreased to 41 kPa (6 psi), after which 200 repetitions of load are applied and deviator load and vertical recoverable deformation at or near the 200th repetition are recorded. At the completion of the load, the chamber pressure is reduced to zero.

Data and Calculations

Test data can be recorded and calculations performed using a table such as illustrated in Figure 10.16. For specific procedures refer to Soils Manual, MS-10, Asphalt Institute.

Soil Sample ____________________
Location ____________________
Sample No. ____________________
Specific Gravity ____________________

Soil Specimen Measurements
Diameter Top ____________________
Middle ____________________
Bottom ____________________
Average ____________________
Membrane Thickness ____________________
Net Diameter ____________________
Ht Specimen + Cap + Base ____________________
Ht Cap + Base ____________________
Initial Length L_0 ____________________

Soil Specimen Weight
Initial Wt of Container
+ Wet Soil-gms ____________________
Final Wt of Container
+ Wet Soil-gms ____________________
Wt Wet Soil Used ____________________

Soil Specimen Volume
Initial Area L_0
cm^3 ($in.^3$) ____________________
Volume $A_0 L_0$
cm^3 ($in.^3$) ____________________
Wet Density-kg/m^3 (lb/ft^3) ____________________
Water Content-% ____________________
% Saturation ____________________
Dry Density-kg/m^3 (lb/ft^3) ____________________

Date ____________________
Compaction Method ____________________
Vertical Spacing Between
LVDT Clamps-mm (in.) ____________________
Chamber Pressure-kPa (psi) ____________________

Constants
Vertical LVDT ____________________
Load Cell ____________________
Comments ____________________
____
____
____
____
____
____

Specimen No.	Load Cell Chart Reading	Deviator Load kg (lb)	σ_d kPa (psi)	Recoverable Deformation LVDT Chart Reading	Recoverable Deformation mm (in.)	ϵ_r mm/mm (in./in.)	$M_r = \sigma_d / \epsilon_r$ kPa (psi)

Figure 10.16 Form for Resilient Modulus Tests on Compacted Fine-Grained Soils.

10.3 Drainage of Asphalt Pavement Structures

Introduction

Reasons for Drainage

Highway engineers recognize the critical need for good drainage in designing and constructing pavements. Probably no other feature is as important in determining the ability of a pavement to withstand the effects of weather and traffic, and in providing trouble-free service over long periods of time.

Water in a subgrade usually is slow to evaporate or drain. Even in dry weather it may remain indefinitely. Often, depressions in porous material become subterranean reservoirs unless they are drained.

Most subgrade soils contain silt and clay, which weaken when wet. This weakening is hastened by vibrations from traffic. If the subgrade is not properly shaped and sloped for drainage, water may collect on its surface and beneath the pavement layers. Eventually this causes the subgrade to lose stability and support. Under traffic, wet soil may be worked into a mud slurry that pushes up into voids in the aggregate base course, thus destroying the grain-to-grain interlock.

Design

In developing the various features of a drainage system, consideration should be given to its principal purposes, which are:

- To collect and drain surface and subsurface water
- To prevent or retard embankment erosion
- To intercept water from surrounding areas and carry it from the area
- To lower the groundwater table

Drainage problems are either surface or subsurface. Each requires separate analysis and treatment.

Surface drainage includes the disposal of all water present on the surface of the pavement and nearby ground. Surface water should be prevented from entering the subdrainage system. Inevitably, some water enters the pavement structure, but may be kept to a minimum by surface maintenance procedures.

Subsurface drainage deals with water in the pavement courses and in the surrounding soil. Such water may enter the pavement through open joints, cracks, and other openings in the surface, through adjacent porous material, or from the ground below.

Subsurface water may be present in several forms, including:

- Water that is free to flow under the force of gravity, referred to as "free water"
- Water that moves by capillary forces, referred to as "capillary water"

- Water in the form of water vapor

Important differences exist between surface and subsurface drainage. Surface runoff is usually a direct result of rainfall or melted snow. Water flow to a subdrain depends entirely on the permeability of the soil and the pavement courses through which it passes. The maximum flow of surface water usually occurs approximately an hour after a rainstorm. In contrast, the maximum flow in a subdrain (not open to surface water) may occur several hours or even days after a heavy rain. Design requirements in one case are based on the amount of rain or melted snow, and in the other case on the permeability of the soil or aggregate.

Surface Drainage

Design for Roadways

Protecting pavement from the action of surface water is most easily accomplished by a full-width asphalt surface. This means that the exterior of the pavement, including the shoulders, is constructed to provide an impervious cover that prevents surface water from entering the pavement structure from the sides. A pavement surface should be free from holes and cracks, have a permanently tight joint with the shoulder or gutter, and be shaped and sloped (crowned) for adequate runoff. Surface water usually is collected at the edge of a paved surface in ditches or gutters (Figures 10.17 and 10.18) which carry it to lower ground or to catch basins and storm sewers. On embankments, water should be directed to asphalt spillways by way of asphalt curbs or dikes constructed on the extreme edge of the shoulder.

If a full-width surface is not used, maintaining a tight joint between the shoulder or gutter may become a problem. Some consideration should be made for removing water that may leak through the joint.

Highway pavements are built with a crown or slope to facilitate the flow of water from their surfaces. The most common design practice allows the surface water to flow across the shoulder into a ditch or gutter at the side.

The most effective shoulder drain is one that has been waterproofed with an asphalt surface.. A less effective practice (but still acceptable) is to cover road shoulders with an aggregate graded to minimize seepage into the subgrade. Where a crown section is used for rapid drainage, the shoulder slope must be steeper than that of the roadway surface. But on a pavement uniformly sloped either to the outside or the inside, such as superelevated curve, the shoulder slope is generally the same as that of the pavement. Sodded shoulders tend to prevent erosion but they may also prevent water from draining freely from the road surface thereby increasing the amount of moisture entering the ditch side slope. On the other hand, grass and roadside vegetation can remove moisture from the soil through its root system.

In some cases, a raised lip or curb 100 mm (4 in.) or more in height may be constructed at the outside shoulder edge to intercept water and guide it to

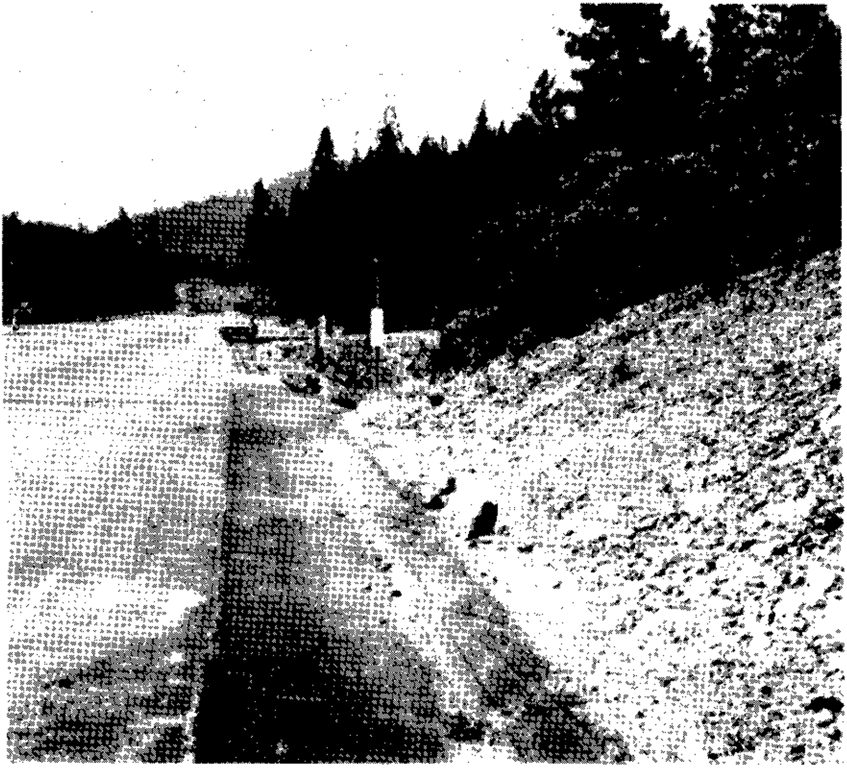


Figure 10.17 Typical Side Ditch.

some convenient outlet. In this design the edge of the pavement is sometimes shaped into a shallow channel (Figures 10.19 and 10.20), which leads into storm sewer openings or other paved channels. Curbs and dikes should be located so that they do not interfere with the removal of snow from the pavement surface.

Drainage ditches receive water from pavement surfaces and from subsurface drains. Where the surrounding area slopes toward the roadway, these ditches also intercept water that would otherwise reach the roadbed. It is usually preferable to widen rather than deepen a drainage ditch to increase its capacity. Side slopes usually vary from about 4:1 on the roadway side to about 8:1 or flatter for the outside slope. The ditch should be large enough to accommodate an estimated surface runoff. It should be constructed on a continuous smooth grade without depressions or pockets that will collect water.

Where the surrounding area consists of higher ground, as in a cut section, water may flow toward the roadway either on the surface or at shallow depths through water-bearing strata. A ditch constructed at the back of the top of the cut, or on benches in the cut slope, will intercept and carry the

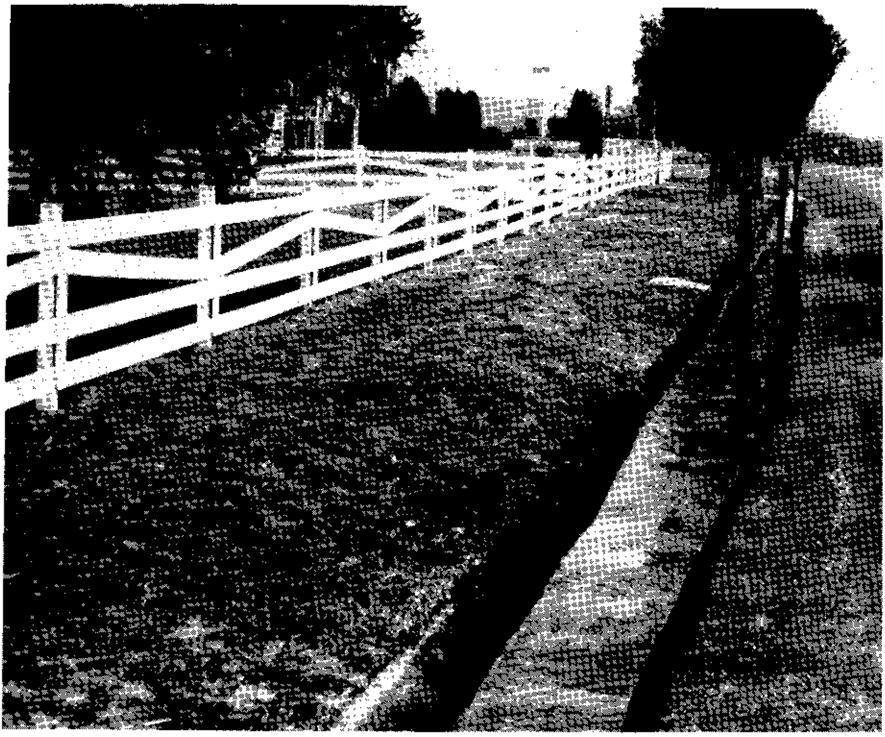


Figure 10.18 Paving Side Ditches Prevents Ditch Erosion.

Figure 10.19 Dike Leading to Discharge Pipe to Carry Water Down Slope Will Generally Reduce Maintenance of Slopes and Ditches.

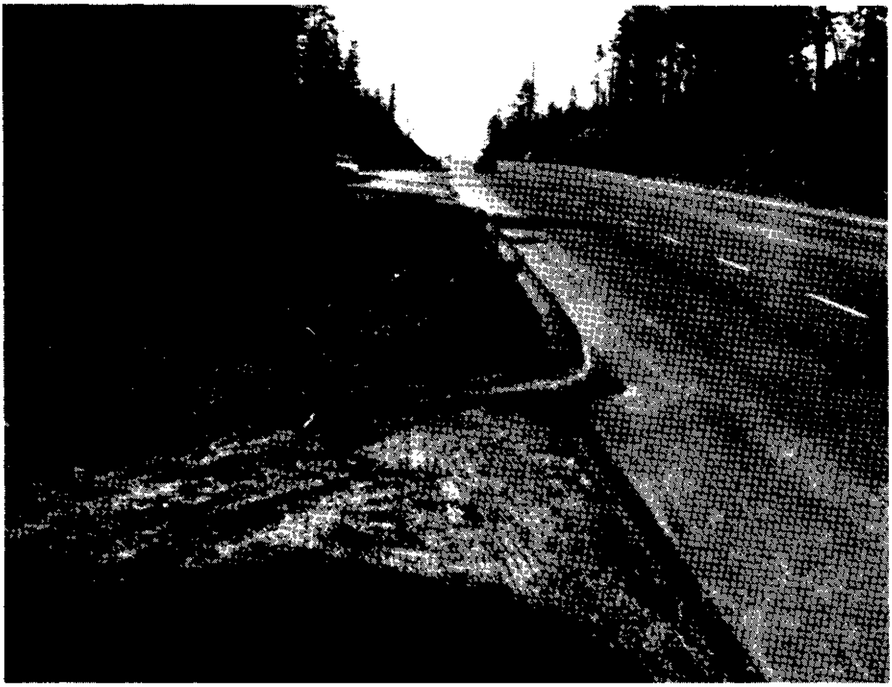


Figure 10.20 Roadside Dike Formed Into Paved Ditch.

water away. Ditches of this type are also effective in preventing slope erosion or sloughing.

Water collected in ditches may discharge directly into natural channels or into storm sewer openings. In other cases, it may be directed from the roadway by special ditches constructed for that purpose.

The ideal velocity of flow in a ditch is one that is fast enough to prevent settling of suspended material but not so fast as to cause erosion or scouring. Generally, a figure of 0.6 m/s (2 fps) for newly seeded loam or fine sand, to 1.8 m/s (6 fps) for well-sodded or coarse gravel linings is an acceptable range of safe velocities. Higher velocities require paving to prevent erosion. Hot-mix asphalt is a desirable material for ditch paving.

Design for Large Paved Areas

Surface drainage design procedures for parking lots are the same as those for airports. A recommended guide, therefore, is the U.S. Federal Aviation Administration's publication *Airport Drainage.*

After the parking area plan has been established and an efficient parking pattern worked out, drainage grades that obviate patrons stepping from cars into gutters are designed.

To prevent puddles from forming in rainy weather, a minimum of 2 percent or 20 mm/m (¼ in. per ft) grade is necessary. On sloping terrain, constructing a grade presents no problem; but in flat areas, built-in drainage valleys give rise to a gently undulating flow on the paved area. Flows to proposed inlets should be calculated for existing runoff conditions and, if necessary, spacing should be adjusted to give satisfactory results. For average conditions, inlet spacing of 60 to 120 m (200 to 400 ft) is satisfactory.

Since only a fixed contour can be laid in one pass of a paving machine, design calling for a meandering gutter or valley should be avoided. Such gutters require hand placing, the quality of which is inferior to machine laying. A layout should provide for drains that follow straight lines. When the mix is being placed by the paving machine, the direction of the paver should parallel these lines. For the most efficient paver use, these lines should parallel the long dimensions of the lot. Watertight longitudinal joints are essential to prohibit the intrusion of surface water and to minimize the possibility of erosion under traffic.

Subsurface Drainage

Introduction

Subsurface water results from rainfall or melting snow soaking into the soil and percolating downward until an impervious layer is reached. At this point the water may form a subsurface pond in the void spaces of the soil. The level of the subsurface water is the ground water table. Subsurface water may also percolate long distances through a porous layer if confined by impervious layers or strata.

A porous or cracked pavement may permit rain water or melted snow to enter the structural section and saturate the various layers. This water may then drain down grade and create problems in other areas. These problems may be avoided most effectively by proper attention to pavement mix design and construction. A dense asphalt base is particularly effective. If the structural section contains open-graded layers, they should be drained by pipes or other positive means to prevent ponding of water.

Subsurface seepage, under certain conditions, may develop a hydrostatic head sufficient to lift the pavement completely off the base, causing cracking and, in extreme cases, complete disintegration of the pavement structure. The problem is more acute when steep grades are involved. With steep gradients the water may travel longitudinally in an untreated base course along the direction of the gradient and cause excessive hydrostatic pressures at sag vertical curves or on the low side of superelevated horizontal curves. In many cut sections there is a need also for longitudinal subdrains to prevent water from higher ground accumulating under the pavement. If not intercepted, subsurface water from a cut may flow out on to a fill and cause slumping of the fill slope and cracking of the pavement. The choice of filter material and the design of the drainage system must be given careful attention, considering both the type of material to be drained and the quantity of water to be expected.

Filter Material and Drain Rock

Filter material may be a granular material having a grading that will prevent migration of adjacent soil particles into the drain; however, the permeability may be low. Drain rock, on the other hand, consists essentially of one-size material which results in very high permeability. Drain rock, however, is subject to clogging unless used with filter material or filter fabrics.

Since highways are constructed with very flat transverse slopes, these reduce to an insignificant amount the water that will drain through a filter material.

The permeability of a granular filter material is reduced very rapidly as the percentage of fine material is increased. This is shown quite clearly by the grading curves shown in Figure 10.21. The change in grading shown on this chart is less than the normal variation which occurs during the production of materials. Yet, one of those materials has eleven times the permeability of the other. Despite the foregoing, it is a fact that in most cases the granular materials available near a construction site are usually on the "dirty" side with respect to the amount and character of fines. And engineers are charged with using these materials in their natural state or modifying them by economical means to make them usable. The logical and practical answer to this problem, however, is to use such materials, with asphalt, in Full-Depth construction. This reduces the possibility of these layers acting as reservoirs for moisture.

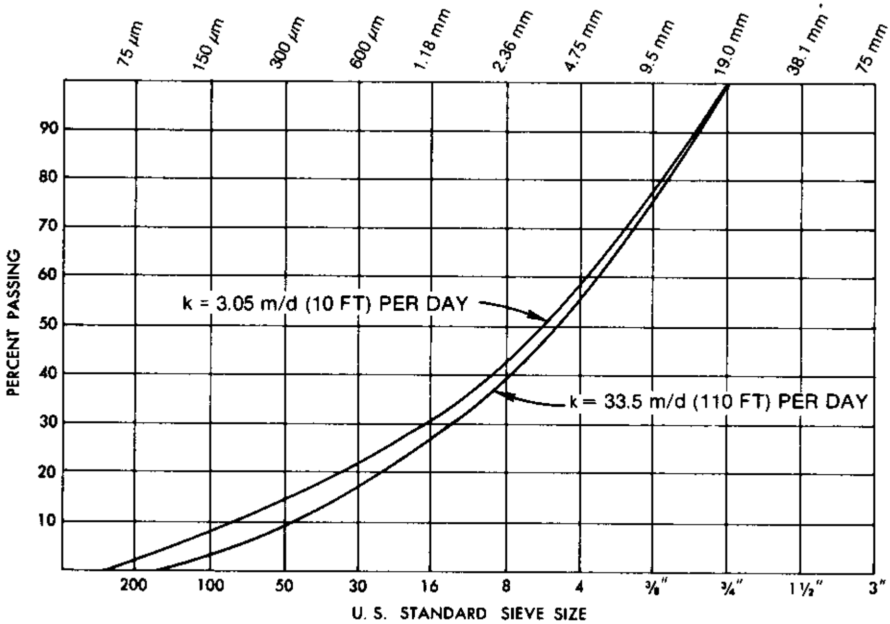


Figure 10.21 The Effect on Permeability of a Minor Change in Grading.

Subsurface Drainage Schemes

The following approaches are suggested for consideration.

(a) *Wet Cuts*

If the material encountered at grade is stratified, it may be possible to remove seepage water by interceptor drains at the toe of the cut slope. A weakness of this type of construction, however, is that water frequently will bypass the drain and rise to the surface beneath the pavement. This is caused by variations or curvature in the stratifications of shear zones brought about by old earth movements.

If an appreciable gradiant is involved, a cross-drain should be placed at the downhill end of the cut. This will intercept any water flowing longitudinally which if not drained could saturate the fills and cause slumping of the fill slopes.

(b) *Pavement Permeability*

A properly-designed and compacted dense-graded asphalt concrete pavement will be impermeable to water.

(c) *High-Ground Water Table*

In instances where the ground-water table is high, deep vertical trenches may be placed on either side of the roadway. Other solutions are to raise the grade of the highway, or construct a Full-Depth asphalt pavement.

Drainage Related to Frost Action

Where the frost depth exceeds the thickness of the pavement structure, a subsurface drainage system may be required to lower the water table to prevent the formation of ice lenses and the damaging effects of frost heaves.

(a) *Frost Heaves*

Frost heaves are a serious problem. They are the result of ice lenses forming in the soil. The ice lenses grow in a downward direction (as water in a pond freezes, being fed by water from below), see Figure 10.22. For ice lenses to grow in a soil, there must be freezing soil temperatures, a water table close to the frost line, and a soil favorable to the rapid movement of vapor or capillary water. Where freezing temperatures are prolonged and the frost line penetrates deep into the soil, the ice lenses that are formed cause a heave at the surface.

To avoid damage to drainage structures it is necessary for design purposes to determine the depth of frost penetration. The most reliable information is obtained from local records of depth of frost penetration in the area and for the soils concerned. Where such local information is not available frost penetration can be estimated from charts obtainable from the U.S. Department of Commerce, National Oceanic and Atmospheric Administration, National Climatic Data Center, Asheville, North Carolina 28801.

Problem soils should be removed to a depth equal to at least half, and preferably three-quarters, of the normal depth of frost penetration for the area concerned.

An alternative method of construction is to remove, mix, and recompact a suitable depth of subgrade to promote uniformity. This method will

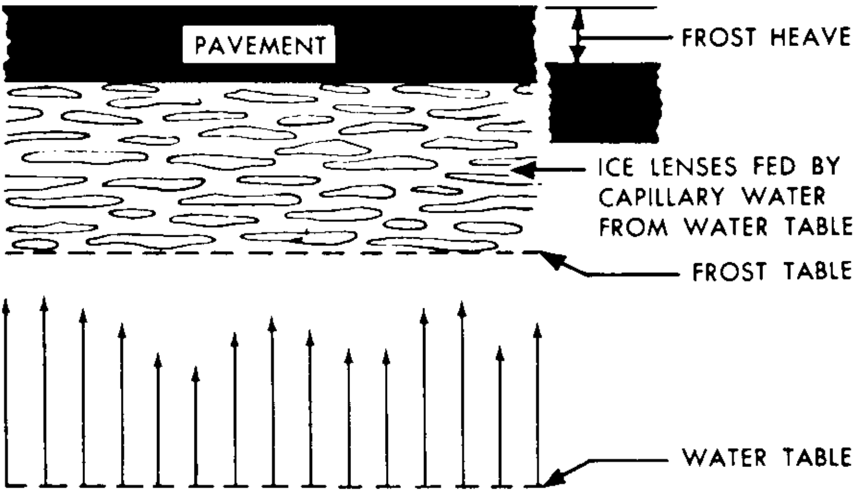


Figure 10.22 Frost Heave is Caused by Ice Lenses Forming Beneath the Pavement Structure.

eliminate differential heaves although the whole road still will rise and fall.

Even if every care is taken, a generalized form of heave can still occur. Frequently, ice accumulates between the granular base and the asphalt layer. This is probably caused by the migration of water vapor from the warm areas deep in the structure toward the colder surface layers. The sudden change in permeability at the asphalt layer restricts the flow of vapor causing it to condense and freeze.

In some instances a lens of ice may form directly under the asphalt layer. The best known means of preventing this is to implement Full-Depth asphalt construction.

(b) *Spring Thaw*

The worst aspect of frost action is the weakening of the subgrade during the thawing period. The soil and moisture conditions which are critical for heaving also are just as severe for thawing damage. The greatest loss of subgrade support is encountered when a wet autumn is followed by a winter with many freeze-thaw cycles and then by a rapid spring thaw.

Another adverse effect of the spring thaw may occur in granular bases over fine-grained subgrades. The fines are carried into the granular base, transforming it from a satisfactory base material into one of low load-carrying capacity. A sand filter blanket, approximately 150 mm (6 in.) in thickness, or a filter fabric may be appropriate under these conditions.

Since frost action sometimes extends into the subgrade, both the heaving and thawing phases often lead to cracking of the pavement and surface. Therefore, filling the cracks to keep water out of the base and subgrade is an important yearly maintenance task. As discussed under Frost Heaves, the effects of spring thaw can be minimized by the use of asphalt Full-Depth pavement and by maintaining proper drainage.

Geotechnical Fabrics

Geotechnical fabrics are manufactured from a number of different materials including polypropylene, polyester, nylon, polyethylene, and polyvinylidene chloride. The function of fabrics as filters is to allow removal of ground water without the build-up of excessive seepage forces or water pressures. The fabric must also prevent piping or subsurface erosion of the soil. In these applications, water flows across the filter into a water-conducting medium, which is usually a trench filled with a selected drainage aggregate and a slotted or perforated pipe which quickly removes the water.

The use of geotechnical fabrics for subsurface drainage may be categorized under the following three general applications.

(a) *Trench Drains*

Fabrics are used as an envelope surrounding the single-size aggregate in the trench. This is to prevent soil from moving into and eventually clogging the stone while allowing the water to pass through the fabric. A more efficient system is provided by installing drain pipes in the bottom of the aggregate-filled trench rather than relying solely on the porous aggregate to discharge the runoff.

(b) *Pipe Wrapping*

Fabrics can also be used as a filter around a drain pipe, particularly for subsurface drainage in sandy or gravelly soils. In the presence of finer grained soils such as silty clays, this type of subdrain has reduced efficiency.

(c) *Base Course Drainage*

Fabrics have been successfully used as a replacement for aggregate filters preventing the migration of fines into untreated aggregate layers. In this type of installation, the fabric prevents the buildup of hydrostatic or seepage pressures.

In the design of subsurface drainage systems, fabrics should be given consideration as an alternative design. Because of the relative ease of installation as compared to the trench placement of a filter aggregate and a coarser aggregate, in separate layers without contamination, the filter fabrics may be more cost effective. A more detailed discussion of the use of geotechnical fabrics is presented in a report prepared by the Federal Highway Administration, FHWA/RD-80/021.

Construction

Drainage Channels

Effective drainage during construction frequently eliminates costly delays as well as later failures resulting from a saturated subgrade. Drainage channels should be constructed early in the grading operations before damage from erosion occurs. Slopes should be protected from erosion as early as practical to lessen the risk of eroded soil choking off existing and newly constructed channels and drainage structures. An effective way of providing protection is to use an asphalt mulch treatment or a sprayed asphalt membrane.

Asphalt-concrete lined channels should be finished to the specified grade and cross section. They should have the specified thickness, though a variation of up to 15 percent may permitted. The linings should be as thick as necessary for the use and the stresses involved, but practical experience indicates that a finished thickness of 50 mm (2 in.) of asphalt concrete should be the minimum thickness specified.

Slide slopes should be as flat as is consistent with other construction requirements, with a maximum slope of 1½:1 permissible up to 3 m (10 ft) depths.

The subgrade should be thoroughly cleaned of all organic and loose material. If conditions are favorable to growth of weeds and other plants in the subgrade, it will be necessary to treat the soil with a soil sterilant prior to placing the asphalt lining.

Soil should be compacted sufficiently to attain stability, particularly on the side slopes. It also should be trimmed as necessary before, during, or after compaction to make it smooth and uniform.

Pipe Installations

In order for drainage systems to operate as designed, all ditches, channels and trenches must be excavated true to line and grade. The bottom quadrant of the barrel of the pipe should be fully and uniformly supported on a stable foundation. It is good practice to excavate a few millimetres (inches) below the grade line and use a material such as gravel, crushed rock, or sand, to bed the pipe. Figure 10.23 shows typical conduit beddings.

All joints should fit tightly and be centered. Pipe sections should be joined in accordance with manufacturers' recommendations.

The strength of any type drainage is to a large extent dependent upon proper backfilling. To obtain maximum load-bearing capacities and to prevent washing out and settlement, it is necessary that the backfill be of a good material, properly placed and carefully compacted. Backfill should be placed carefully and compacted in 150 mm (6 in.) layers uniformly on both sides of the pipe. For pipes smaller than 300 mm (12 in.) in diameter, the first layer should not be deeper than half the diameter of the pipe. This method of backfilling should be continued to a point at least 300 mm (1 ft) above the top of the pipe. Above this point, the layers should not exceed 200 mm (8 in.) and should be compacted to the required density.

Trench Construction

The construction of trench drains is influenced by existing conditions, such as the type of soil and the amount of water present as well as the function the drain must perform. If possible the drain should be designed for ease of construction with equipment normally available on a construction project.

For shallow drains 0.3 to 0.6 m (1 to 2 ft) deep—designed to remove water from the base course or a drainage layer under the pavement—a V-trench is generally the most practical. The shape more nearly fits the flow pattern of water to be removed, and placement of pipe and backfill material is

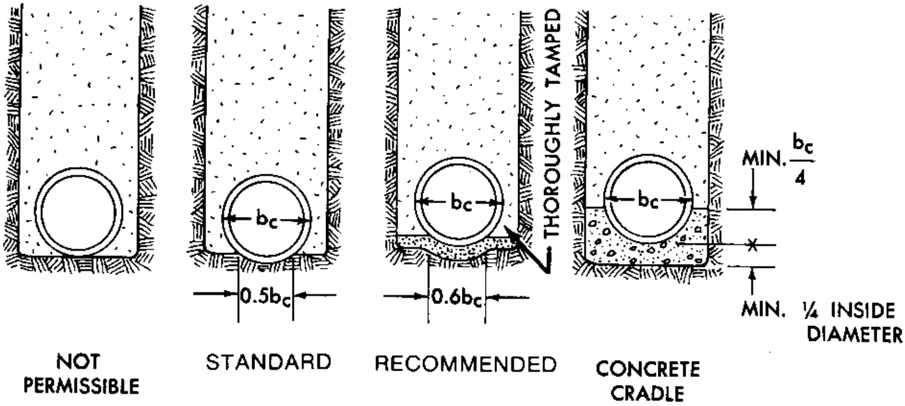


Figure 10.23 Typical Bedding for Trench Conduits.

facilitated. This is particularly valid when the design requires a two-layer drain. A V-trench is also required in waterbearing sand because such material will not stand on steep slopes. The trench may be constructed with a motor grader—a piece of equipment normally available.

For deep drains designed to intercept water from a pervious layer, or to lower the water in a uniform soil, a trench with vertical sides (because it requires less excavating) will usually be most economical. Shoring is required in unstable formations, however, if the depth exceeds 0.6 to 0.9 m (2 to 3 ft). If a two-layer drain is necessary, the hand placing of backfill will be required in order that mixing the two materials can be avoided.

Checking Subsurface Drainage During Construction

Regardless of the care used in the preliminary investigation, soil survey, and design of the highway, it usually is not possible to determine exactly, from borings, the elevation of waterbearing strata or the rate of flow that will be developed. For this reason, it is essential that the need for, and the adequacy of, any subsurface drainage indicated on the plans be continually evaluated.

One should not be misled by the seasonal variations in seepage common to many areas, but should make a careful study of the highway. Consideration of geologic features will generally help in this evaluation. Normally, seepage from deep strata is less affected by seasonal variation than drainage from the lower levels of terrace gravels exposed at the surface.

Any wet, soft, or spongy areas encountered at grade should be investigated and provisions made for their proper drainage. Even a minor rate of seepage can build up a large quantity of water over a period of time if a means of escape is not provided.

Chapter 11

Structural Design of Asphalt Pavements

Understanding the design principles of the asphalt pavement structure, whether for a roadway or an airfield runway, will benefit all those associated with pavement design, construction and maintenance. The procedure described here is based on an application of elastic theory to pavement design that utilizes the results of currently acceptable research, advances in asphalt paving technology, existing knowledge of material properties, and present needs of our transportation systems.

SECTION 11.1 Pavement Design Background
- ☐ Asphalt Pavement Structure
- ☐ Full-Depth Asphalt Pavements

SECTION 11.2 Structural Design Methods
- ☐ General Procedure
- ☐ Subgrade Resilient Modulus

SECTION 11.3 Traffic Analysis and Thickness Design - Highways
- ☐ Traffic Analysis
- ☐ Thickness Design

SECTION 11.4 Traffic Analysis and Thickness Design - Airfields
- ☐ Aircraft Analysis
- ☐ Thickness Design

SECTION 11.5 Traffic Analysis and Thickness Design - Heavy Wheel Loads
- ☐ Vehicle Analysis
- ☐ Thickness Design

SECTION 11.6 Planned Stage Construction

☐ Advantages
☐ Design Approach

SECTION 11.7 Pavement Overlay Design

☐ Evaluating Structural Adequacy
☐ Effective Thickness Analysis
☐ Overlay Thickness Determination

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COMPUTER SOFTWARE

1. *Computer Program DAMA, CP-1,* Asphalt Institute.
2. *Computer Program Heavy Wheel Load, CP-2,* Asphalt Institute.
3. *Computer Program Airport, CP-3,* Asphalt Institute.
4. *Asphalt Design Thickness for Highways & Overlays, CP-4,* Asphalt Institute.
5. *Life Cycle Cost Analysis, CP-5,* Asphalt Institute.

11.1 Pavement Design Background

An adequate and economical design for a pavement structure is just as important as a design for any other engineering structure. An under-designed pavement will fail prematurely, costing more money for repairs. A wasteful over-design, or a selection of materials that are not the most economical and suitable for the design, is contrary to the standards of sound engineering and will unnecessarily deplete a limited budget.

Asphalt Pavement Structure

An asphalt pavement structure consists of courses of asphalt-aggregate mixtures plus any untreated aggregate courses between the asphalt construction and the foundation or subgrade. To lessen the chances of future maintenance problems, the most cost effective process is to properly grade and compact the soil foundation before building the pavement. Density requirements vary between highways, airfields, and heavy wheel load types of facilities. Therefore, the specific design manuals should be checked for the recommended level of subgrade preparation to be performed.

The subgrade ultimately carries all traffic loads. Therefore, the structural function of a pavement is to support a wheel load on the pavement surface and spread and transfer that load to the subgrade without exceeding either the strength of the subgrade or the internal strength of the pavement itself (Figure 11.1a).

In Figure 11.1a, wheel load, W, is transmitted to the pavement surface through the tire at an approximately uniform vertical pressure, P_o. The pavement then distributes the wheel load over a larger area of subgrade so the maximum pressure on the subgrade is only P_1. By proper selection of pavement materials and with adequate thickness of pavement, P_1 will be small enough to be easily supported by the subgrade. Figure 11.1b shows the general manner in which the maximum vertical pressure intensity decreases with depth from P_o to P_1.

Full-Depth Asphalt Pavements

A Full-Depth asphalt pavement is one in which asphalt mixtures are employed for all courses above the subgrade or improved subgrade. Asphalt-treated base layers are placed directly on the subgrade.

Full-Depth asphalt pavements have many advantages, and one of the more important is their ability to resist pavement stresses (Figure 11.2). In Figure 11.2, wheel load, W, deflects the pavement structure causing both tensile and compressive stresses. Asphalt-aggregate mixtures resist these stresses far better than unbonded aggregate layers, which have no tensile strength. Therefore, asphalt bases spread the wheel load over broader areas than untreated granular bases and, as a result, less total pavement structure is required.

In addition, Full-Depth asphalt pavements have other advantages, which include resistance to the destructive effects of groundwater moisture as well as faster, easier construction. On widening projects, where adjacent traffic flow must usually be maintained, Full-Depth asphalt can be especially advantageous. Here, the simplicity and speed of Full-Depth asphalt construction means direct savings in traffic control costs and minimal disruption to users and adjacent businesses.

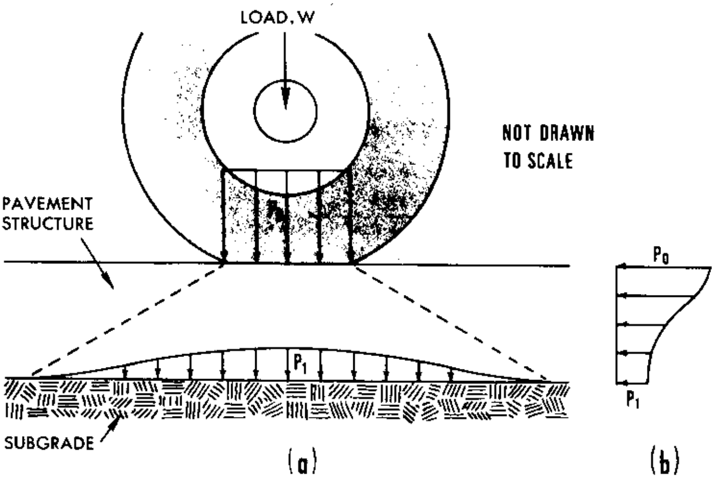


Figure 11.1 Spread of Wheel Load Through Pavement Structure.

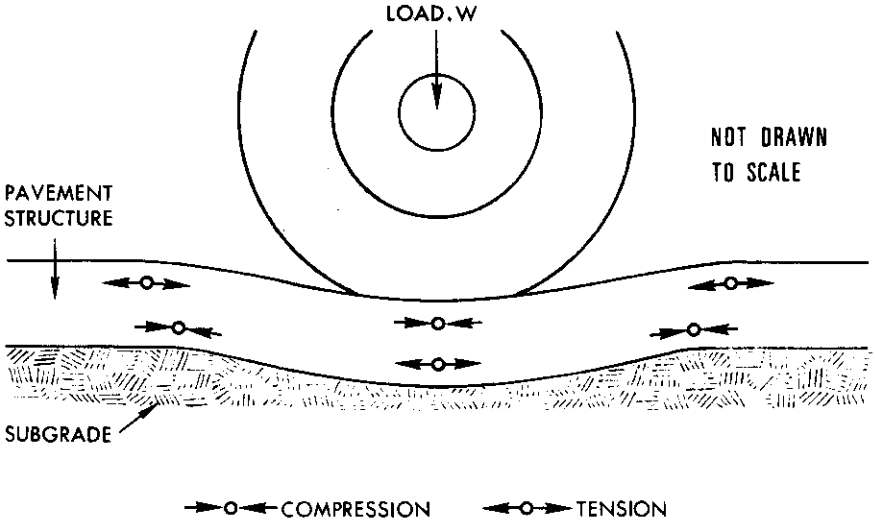


Figure 11.2 Wheel-Load Deflection Results in Tensile and Compressive Stresses in the Pavement Structure.

11.2 Structural Design Methods

General Procedure

The design procedures described in this chapter are used to determine the thickness of pavement structures consisting of asphalt concrete surface or emulsified asphalt surface (with surface treatment) over asphalt concrete base, emulsified asphalt base, or untreated aggregate base or subbase. The common steps in each design procedure are:

1. Select or determine input data:
   (a) subgrade resilient modulus, M_r
   (b) expected traffic
   (c) surface and base material types (refer to Chapter 1)
   (d) climatic (temperature and drainage conditions)
2. Determine design thicknesses for the specific conditions described by the input data.
3. Prepare stage construction design, if appropriate. (see "Stage Construction," this chapter.)
4. Make an economic analysis of the various solutions arrived at for the design problem.
5. Select the final design cross-section.

The specific details of these steps will vary with the type of traffic being supported by the pavement, i.e., highways, airfields, or very heavy wheel loads. This chapter will highlight the procedure to be used for each of these situations, however, the respective manual listed in the bibliography should be consulted for more detail.

Subgrade Resilient Modulus

One of the most important factors in every pavement design is the strength of the subgrade. The design procedures discussed in this Chapter are based on the use of resilient modulus, M_r, to characterize subgrade strength. The method of test for resilient modulus is covered in Chapter 10 and the Soils Manual, MS-10, Asphalt Institute.

M_r can be estimated from California Bearing Ratio (CBR), plate bearing value, and Hveem Resistance (R-value) test data. Procedures for performing these tests are also covered in Chapter 10 and the Soils Manual, MS-10. Approximate values of M_r are obtained as follows. Note, these relationships are *not* applicable to aggregate base materials.

The subgrade resilient modulus may be approximated from CBR test values according to the relationship

$$M_r\ (\text{MPa}) = 10.3\ \text{CBR}$$
$$(M_r\ (\text{psi}) = 1500\ \text{CBR})$$

The subgrade resilient modulus may be approximated from the Plate

Bearing test value by using the correlation chart, Figure 11.3.

The subgrade resilient modulus may be approximated from the R-value test results according to the relationship

$$M_r\ (\text{MPa}) = 8.0 + 3.8\ (\text{R-value})$$

$$(M_r\ (\text{psi}) = 1155 + 555\ (\text{R-value}))$$

To determine the proper M_r for design it is recommended that a minimum of six to eight test values be obtained, depending on the size of the project, and that this procedure be used to determine the design subgrade, resilient modulus, M_r:

1. Test six to eight samples of subgrade. Determine M_r or convert CBR or Plate Bearing Value or R-value test data to subgrade resilient modulus value.
2. Arrange all test values in numerical order.

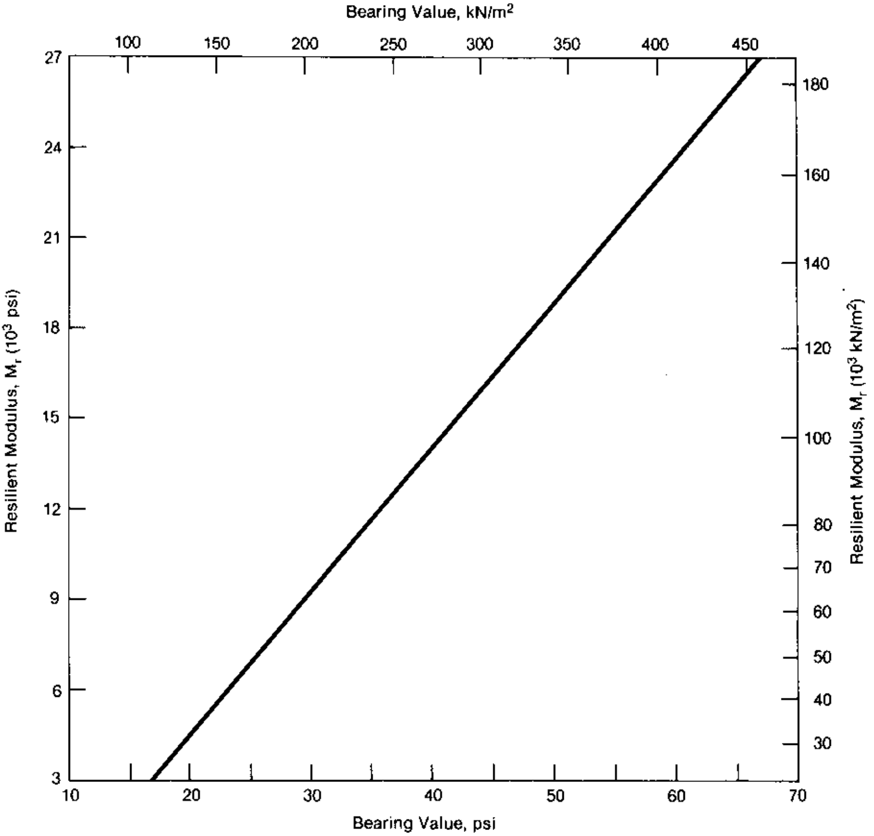


Figure 11.3 Approximate Relationship Between Plate Bearing Value and Resilient Modulus (M_r).

3. For each change in test value, beginning with the lowest value, compute the percentage of the total number of values that are equal to or greater than this value.

4. Plot the results on cross-section paper.

5. Draw a smooth, best-fit curve through the plotted points. (Note: if the test data are well-distributed the curve should be S-shaped and the 50th percentile value should fall close to the average of the data.)

6. Read from the curve the subgrade strength value at the appropriate design subgrade percentile value shown in Table 11.1. This is the design subgrade strength value.

Table 11.1 Subgrade Design Limits

Type of Facility	Traffic Level	Design Subgrade Percentile Value, Percent
Highway (EAL)	10^4 or less	60
	Between 10^4 & 10^6	75
	10^6 or more	87.5
Airfield	All	85
Heavy Wheel	All	85

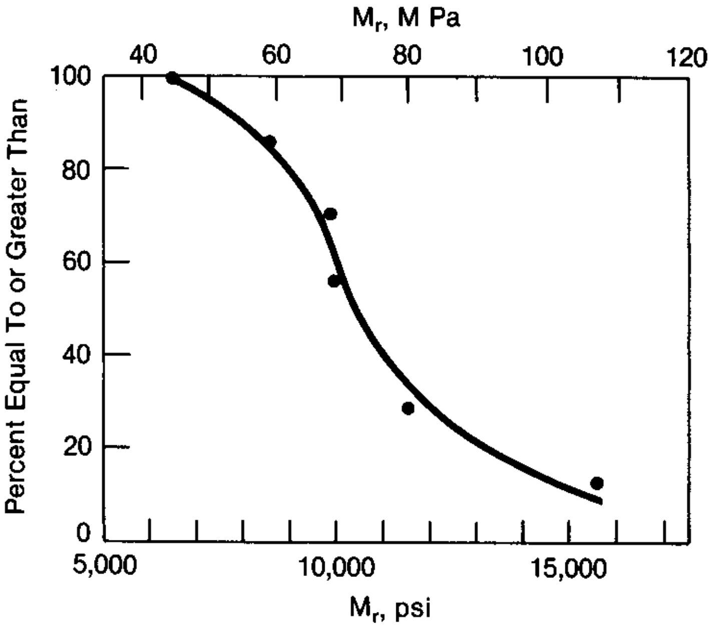
Example—Determine Design Subgrade Resilient Modulus for the conditions given below.

1. Design Traffic EAL (Equivalent Axle Load) = 10^4, 10^5 and 10^6.

2. The results of seven tests produced the following subgrade resilient modulus test values: 44.8, 58.6, 67.6, 68.3, 68.3, 80.0, 106.9 MPa (6,500; 8,500; 9,800; 9,900; 9,900; 11,600; 15,500 psi).

3. Percentile values have been calculated as follows:

Test Values		Percent Equal to
MPa	psi	or Greater Than
44.8	6,500	100
58.0	8,500	86
67.0	9,800	71
68.3	9,900	57
68.3	9,900	57
80.0	11,600	29
106.9	15,500	14



4. Plot test values vs percent equal to or greater than as shown above.

5. Design subgrade M_r values for the different EAL values have been selected from the above plot, and summarized below.

	Design	Design Subgrade M_r	
EAL	**Percentile Value**	**MPa**	**psi**
10^4	60	69.0	10,000
10^5	75	64.1	9,300
10^6	87.5	56.9	8,250

11.3 Traffic Analysis and Thickness Design - Highways

Traffic Analysis

Estimating and Evaluating Traffic

Obviously, no previous traffic data are available that apply directly to a new (soon to be constructed) pavement facility. The designer must therefore depend on traffic studies of similar facilities and community or regional planning studies to assist him in his design.

The traffic analysis used in this procedure requires estimates of the number of vehicles of different types, such as passenger cars, buses, single-unit trucks, and multiple-unit trucks expected to use the proposed facility. Traffic classification counts for particular highways are made periodically by the U.S. Federal Highway Administration, state highway and other agencies. This information should be available and used where applicable in pavement structural design.

When classification data are not available, estimates can be obtained from Table 11.2. This table was compiled from truck-count data collected by the states in cooperation with the U.S. Federal Highway Administration.

Table 11.2 Average Distribution of Trucks on Different Classes of Highways—United States[1]

Truck Class	Percent Trucks[2]					
	Rural Systems			Urban Systems		
	Inter-State	Other Principal	Minor Arterial	Inter-State	Other Freeways	Other Principal
Single-Unit Trucks						
2-axle, 4-tire	43	60	71	52	66	67
2-axle, 6-tire	8	10	11	12	12	15
3-axle or more	2	3	4	2	4	3
All single-units	53	73	86	66	82	85
Multiple Unit Trucks						
4-axle or less	5	3	3	5	5	3
5-axle[3]	41	23	11	28	13	12
6-axle or more[3]	1	1	<1	1	<1	<1
All multiple units	47	27	14	34	18	15
All trucks	100	100	100	100	100	100

[1] Compiled from data supplied by the Highway Statistics Division, U.S. Federal Highway Administration.

[2] Individual situations may differ from these average values by 50 percent or more.

[3] Including full-trailer combinations in some states.

The design lane is the lane in which the greatest amount of truck traffic is expected. This is either lane of a two-lane roadway or the outside lane of a multi-lane highway. In the absence of specific data, Table 11.3 may be used for determining the relative proportion of trucks to be expected in the design lane.

Table 11.3 Percentage of Total Truck Traffic in Design Lane

Number of Traffic Lanes (Two Directions)	Percentage of Trucks in Design Lane
2	50
4	45 (35-48)[1]
6 or more	40 (25-48)[1]

[1] Probable range.

Traffic Loads

Traffic on highways and streets varies both in the number of vehicles and in the magnitude of loadings. Legal axle load limits vary among states and also vary for different classes of roads within many states. Traffic patterns also vary and are constantly changing as a result of land-use developments. Current and future traffic conditions therefore cannot be established precisely.

Nevertheless, the cumulative effects of traffic loads are very important factors in the structural design of a pavement. Both the initial traffic conditions and the manner in which they may be expected to change must be evaluated. Because of the infinite variety of traffic conditions, their cumulative effects must be based on some common denominator for practical use in a pavement design method. Procedures outlined in this section are based on repetitions of an equivalent axle load (EAL) as the common denominator.

The effect on the pavement performance of any combination of axle loads of varying magnitude is equated to the number of 80 kN (18,000 lb) single-axle loads required to produce an equivalent effect. This is known as the equivalent 80 kN (18,000 lb) single axle load (EAL). Extensive studies have provided factors for converting various axle loads to an equivalent number of 80 kN (18,000 lb) single-axle loads (EAL).

The number of equivalent 80 kN (18,000 lb) single-axle load applications contributed by one passage of a single *vehicle* is called the Truck Factor.

Average typical Truck Factors for a variety of U.S. truck and highway classifications are given in Table 11.4.

Traffic Growth

Pavements must be designed to serve traffic needs adequately over a period of years. Traffic growth must therefore be anticipated when determining structural requirements of the pavement. Traffic growth history for comparable facilities, and community and regional planning programs together provide a basis for the estimate. Overall growth does not appear to differ greatly between urban and rural highways. However, since average growth percentages are based on total vehicle miles traveled, it is advisable at times to determine, if possible, separate growth percentages for trucks and passenger vehicles.

Table 11.4 Average Truck Factors (TF) for Different Classes of Highways and Vehicles—(United States[1])

Vehicle Type	Truck Factors[2]					
	Rural Systems			Urban Systems		
	Inter-State	Other Principal	Minor Arterial	Inter-State	Other Freeways	Other Principal
Single-Unit Trucks						
2-axle, 4-tire	0.003	0.003	0.003	0.002	0.015	0.002
2-axle, 6-tire	0.21	0.25	0.28	0.17	0.13	0.24
3-axle or more	0.61	0.86	1.06	0.61	0.74	1.02
All single-units	0.06	0.08	0.08	0.05	0.06	0.09
Tractor semi-trailers						
4-axle or less	0.62	0.92	0.62	0.98	0.48	0.71
5-axle[3]	1.09	1.25	1.05	1.07	1.17	0.97
6-axle or more[3]	1.23	1.54	1.04	1.05	1.19	0.90
All multiple units	1.04	1.21	0.97	1.05	0.96	0.91
All trucks	0.52	0.38	0.21	0.39	0.23	0.21

[1] Compiled from data supplied by the Highway Statistics Division, U.S. Federal Highway Administration.
[2] Individual situations may differ from these average values by 50 percent or more.
[3] Including full-trailer combinations in some states.

Normal growth in the United States is about 3 to 5 percent per year compounded. However, new facilities or new developments can generate increases of greater magnitude. Growth rates changing from 4 to 9 percent have been suggested for U.S. rural highways and from 8 to over 10 percent for some Interstate highways. In applying growth factors, care should be taken that the capacity of the roadway is not exceeded. This impossible growth could result in an unnecessarily conservative design.

Growth may be accounted for in design using the Growth Factors given in Table 11.5. These factors multiplied by the first year traffic estimate will give the total volume of traffic expected during the design period.

The design period is the number of years from the initial opening to traffic until the first major resurfacing or overlay.

Design period should not be confused with pavement life. Pavement life may be extended indefinitely by adding asphalt overlays as required, or until other factors make the pavement obsolete.

Determining Design EAL

The following steps outline the procedure for determining Design EAL:

1. Determine the average number of each type of vehicle expected on the design lane during the first year of traffic.

2. Select from Table 11.4, a Truck Factor for each vehicle type found in step 1.

3. Select from Table 11.5, a single Growth Factor for all vehicles or separate Factors for each vehicle type, as appropriate.

4. Multiply the number of vehicles of each type (step 1) times the Truck Factor and the Growth Factor (or Factors) determined in steps 2 and 3.

5. Sum the values determined in step 4 to obtain the Design EAL.

Figure 11.4 is an example of a worksheet showing the calculation of the Design EAL for a two-lane rural highway following the procedure outlined here.

Table 11.5 Growth Factors[1]

Design Period, Years (n)	Annual Growth Rate, percent							
	No Growth	2	4	5	6	7	8	10
1	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
2	2.0	2.02	2.04	2.05	2.06	2.07	2.08	2.10
3	3.0	3.06	3.12	3.15	3.18	3.21	3.25	3.31
4	4.0	4.12	4.25	4.31	4.37	4.44	4.51	4.64
5	5.0	5.20	5.42	5.53	5.64	5.75	5.87	6.11
6	6.0	6.31	6.63	6.80	6.98	7.15	7.34	7.72
7	7.0	7.43	7.90	8.14	8.39	8.65	8.92	9.49
8	8.0	8.58	9.21	9.55	9.90	10.26	10.64	11.44
9	9.0	9.75	10.58	11.03	11.49	11.98	12.49	13.58
10	10.0	10.95	12.01	12.58	13.18	13.82	14.49	15.94
11	11.0	12.17	13.49	14.21	14.97	15.78	16.65	18.53
12	12.0	13.41	15.03	15.92	16.87	17.89	18.98	21.38
13	13.0	14.68	16.63	17.71	18.88	20.14	21.50	24.52
14	14.0	15.97	18.29	19.16	21.01	22.55	24.21	27.97
15	15.0	17.29	20.02	21.58	23.28	25.13	27.15	31.77
16	16.0	18.64	21.82	23.66	25.67	27.89	30.32	35.95
17	17.0	20.01	23.70	25.84	28.21	30.84	33.75	40.55
18	18.0	21.41	25.65	28.13	30.91	34.00	37.45	45.60
19	19.0	22.84	27.67	30.54	33.76	37.38	41.45	51.16
20	20.0	24.30	29.78	33.06	36.79	41.00	45.76	57.28
25	25.0	32.03	41.65	47.73	54.86	63.25	73.11	98.35
30	30.0	40.57	56.08	66.44	79.06	94.46	113.28	164.19
35	35.0	49.99	73.65	90.32	111.43	138.24	172.32	271.02

[1] Factor $\frac{(1+r)^n - 1}{r}$, where $r = \frac{\text{rate}}{100}$ and is not zero.

If Annual Growth is zero, Growth Factor = Design Period.

Location: Four-Lane Interstate Rural Highway							Design Period: 20 Years
Vehicle Type	Number of Vehicles (per year) 1*		Truck Factor 2		4% Growth Factor 3		EAL [1 X 2 X 3] 4
Single-Unit Trucks							
2-Axle, 4-Tire	84,700	X	0.003	X	29.8	=	7,600
2-Axle, 6-Tire	15,800	X	0.21	X	29.8	=	98,900
3-Axle or More	4,000	X	0.61	X	29.8	=	72,700
All Singles	104,500				Subtotal		179,200
Tractor Semi-Trailers and Combinations							
4-Axle or Less	9,800	X	0.62	X	29.8	=	181,100
5-Axle	80,800	X	1.09	X	29.8	=	2,624,500
6-Axle or More	2,000	X	1.23	X	29.8	=	73,300
All Tractors, Etc.	92,600				Subtotal		2,878,900
All Trucks	197,100				Design EAL = Total		3,058,100

*Based on AADT = 5,000 during first year of traffic, 45 percent in the Design Lane, 24 percent trucks.

Figure 11.4 Example Worksheet for Traffic Analysis.

Thickness Design

Design Charts

The Asphalt Inttitute's *Thickness Design* Manual (MS-1) contains several design charts for these type of cross-sections:

1. Full-Depth Asphalt Concrete
2. Emulsified Asphalt Mixes:
   Type I—Emulsified asphalt mixes made with processed, dense-graded aggregates.
   Type II—Emulsified asphalt mixes made with semi-processed, crusher-run, pit-run, or bank-run aggregates.
   Type III—Emulsified asphalt mixes made with sands or silty sands.
3. Untreated Aggregate Base, 150 mm (6 in.) and 300 mm (12 in.).

Charts 1 to 6 (Figures 11.5 to 11.10) are examples of design charts for Full-Depth asphalt concrete, emulsified asphalt mix Type II, and untreated aggregate base, 150 mm (6 in.) thick. These charts provide the necessary design thicknesses as a function of design subgrade modulus, M_r and design EAL for an average climatic condition [mean annual air temperature of 15.5° C (60° F)].

The design method is also available in the form of a computer program (CP-4) from the Asphalt Institute.

Thickness Determination

The thickness design charts may be used to determine the thickness of one kind of base alone, or for several kinds of bases to compare costs. Therefore, one or more of the following thickness determinations may be performed:

1. Full-Depth asphalt concrete pavements,
2. Emulsified asphalt base pavements with asphalt concrete surface,
3. Pavements with asphalt concrete and untreated aggregate base, or
4. Pavements with emulsified asphalt mixes and untreated aggregate base.

All design charts include a minimum thickness of asphalt concrete surface course. The remainder of the asphalt courses are composed of asphalt concrete base or of emulsified asphalt mixtures. Asphalt concrete surface and base mixtures meeting the recommended mix design criteria have similar structural properties. To provide sufficient durability, however, the minimum thicknesses in Tables 11.6 and 11.7 should meet the mix-design criteria for surface course mixtures.

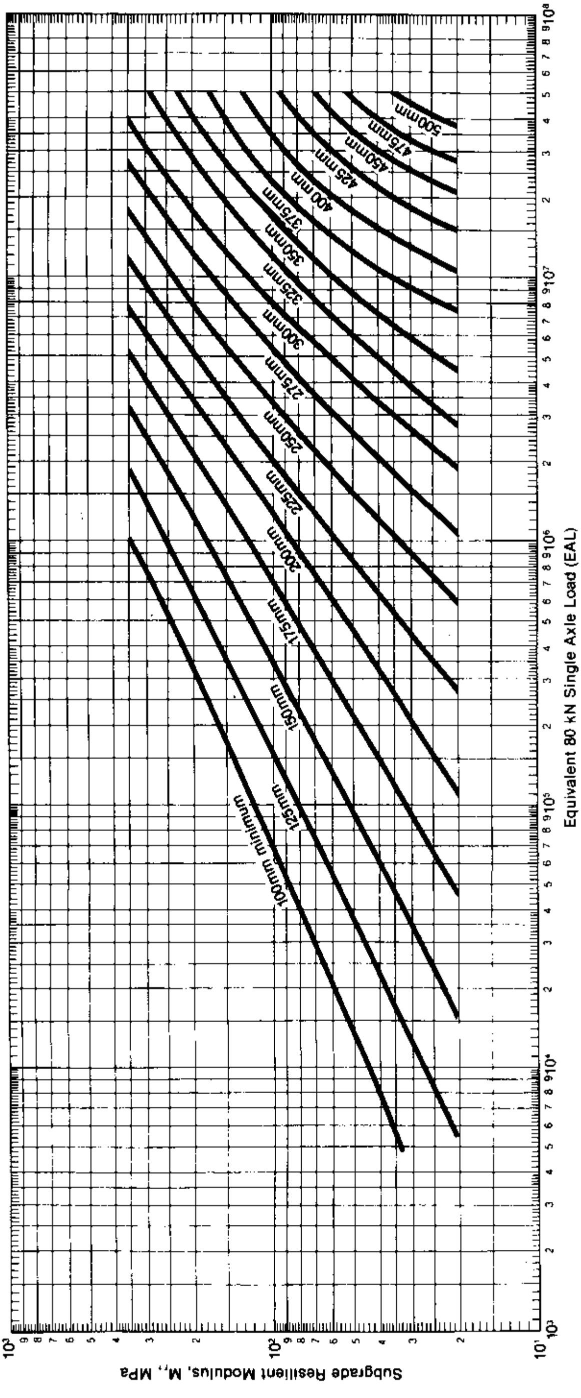


Figure 11.5 Chart 1, Full-Depth Asphalt Concrete (Metric Units).

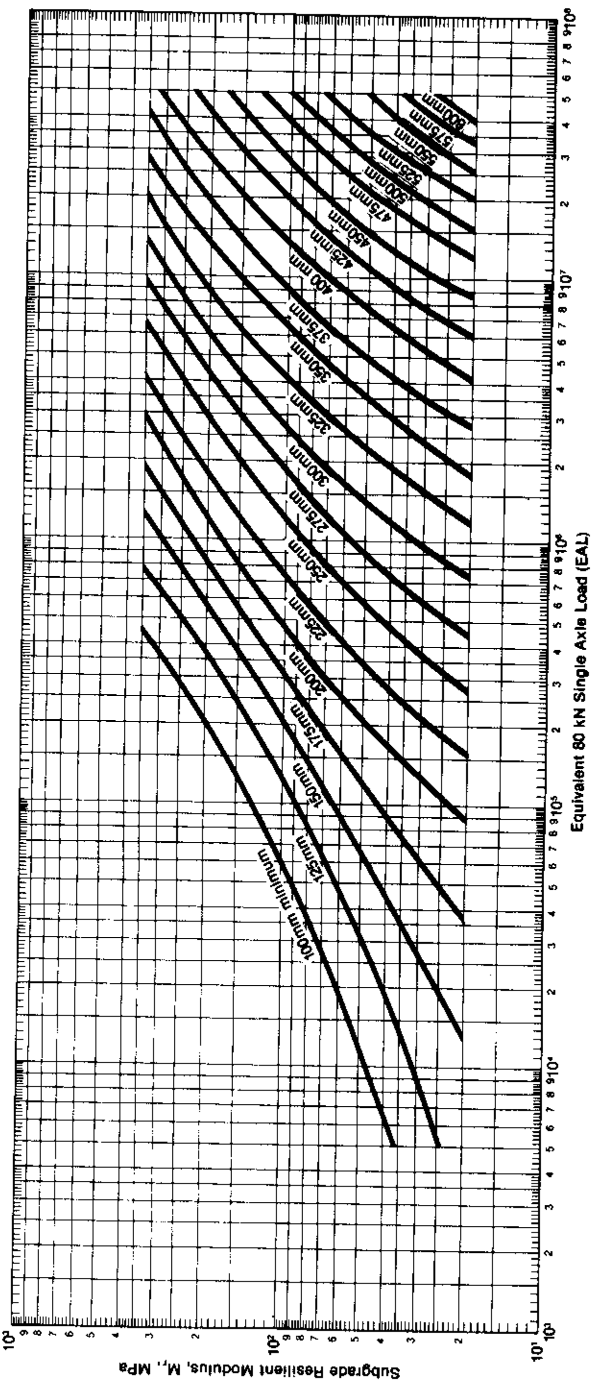


Figure 11.6 Chart 2, Emulsified Asphalt Mix, Type II (Metric Units).

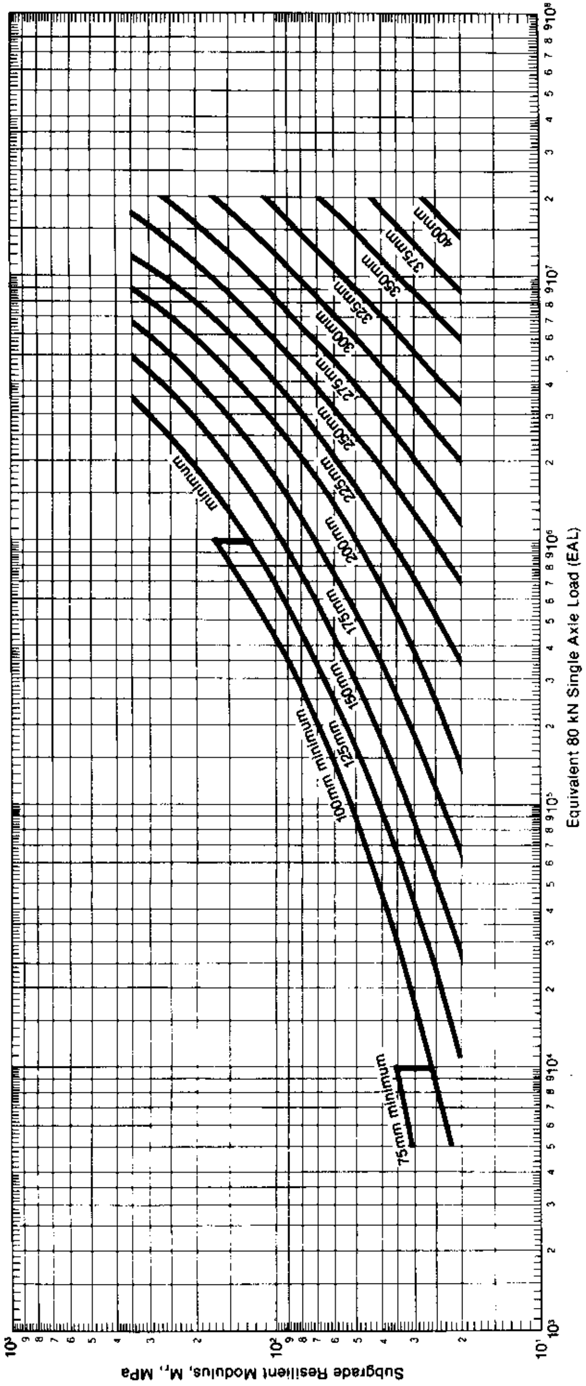


Figure 11.7 **Chart 3, Asphalt Concrete on Untreated Aggregate Base (Metric Units).**

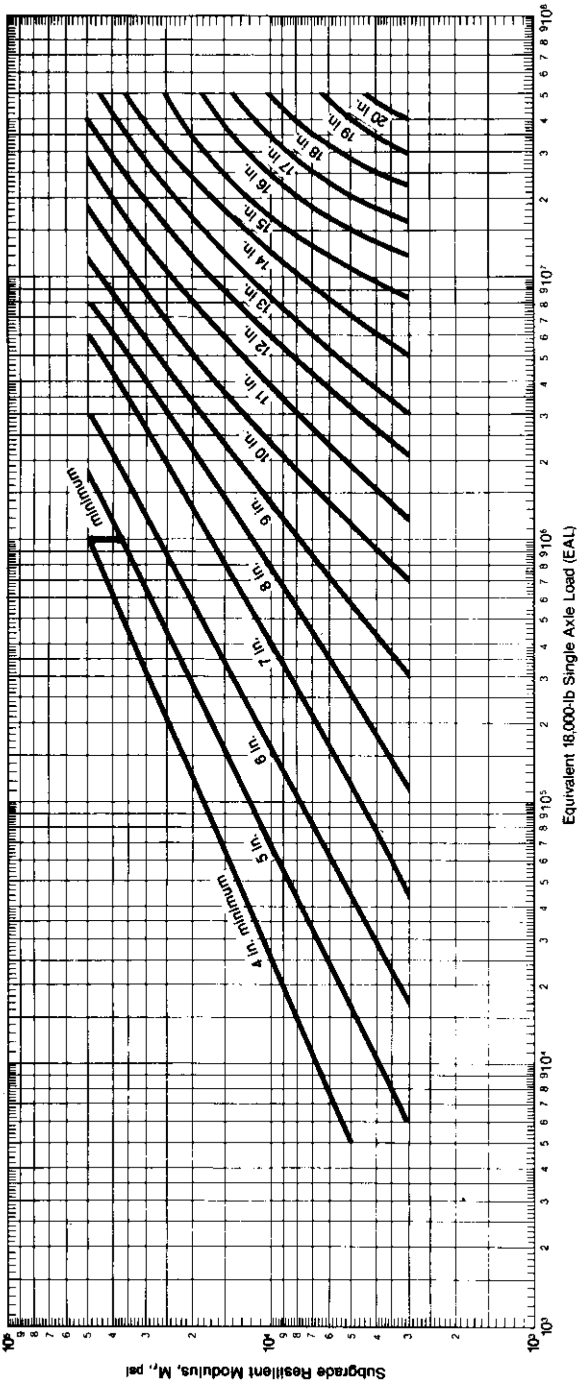


Figure 11.8 Chart 4, Full-Depth Asphalt Concrete (U.S. Customary Units).

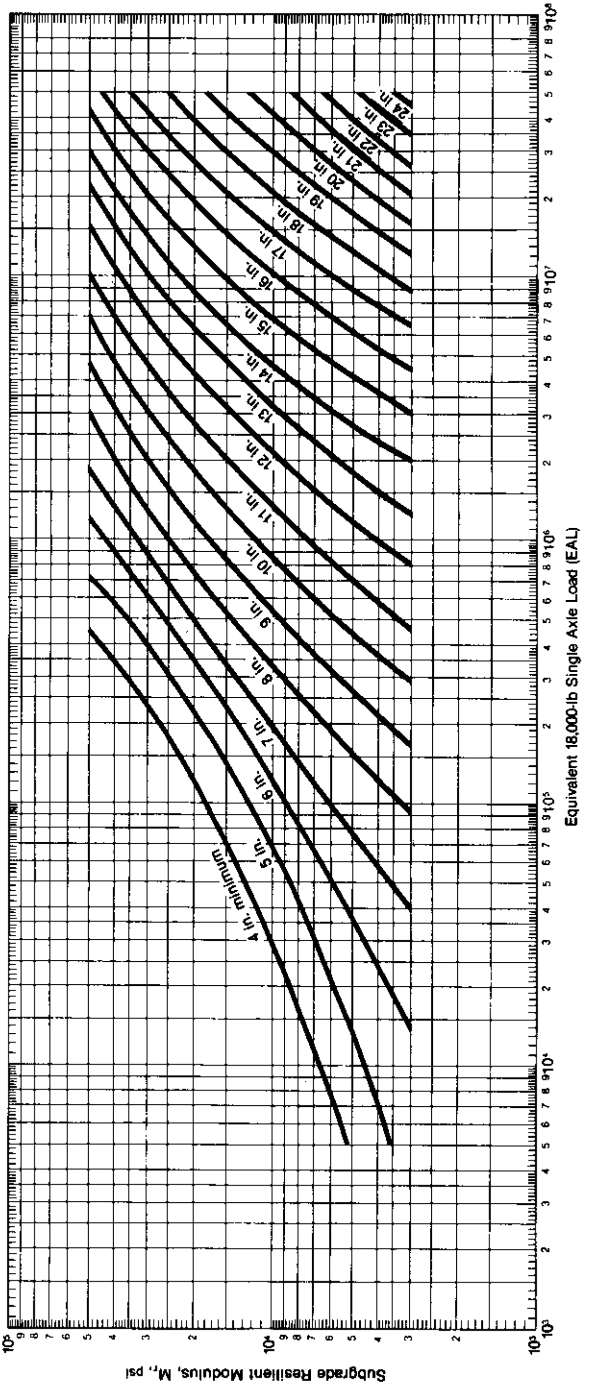


Figure 11.9 Chart 5, Emulsified Asphalt Mix Type II (U.S. Customary Units).

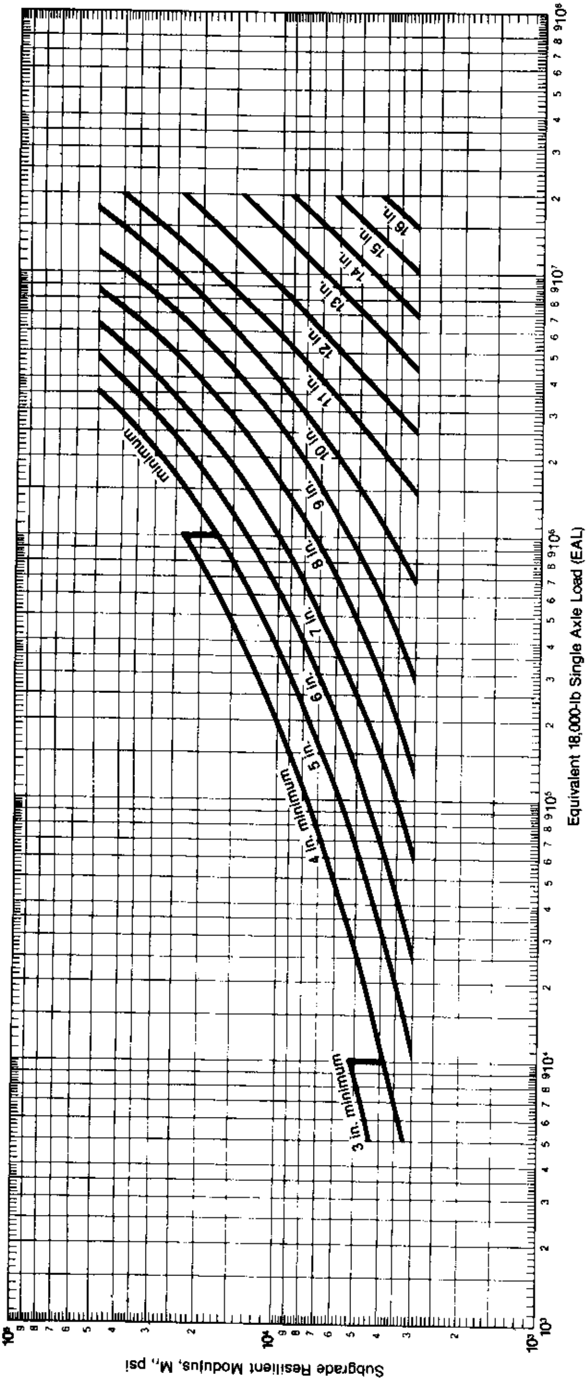


Figure 11.10 Chart 6, Asphalt Concrete on Untreated Aggregate Base (U.S. Customary Units).

Table 11.6 Minimum Thickness of Asphalt Concrete Surface Course

Traffic EAL	Thickness
10^4	25 mm (1.0 in.)
10^5	40 mm (1.5 in.)
10^6	50 mm (2.0 in.)

Table 11.7 Minimum Thickness of Asphalt Concrete Over Emulsified Asphalt Bases

	Thickness of Asphalt Concrete			
	Type I Base		Type II and Type III Base	
Traffic Level EAL	mm	(in.)	mm	(in.)
10^4	25	(1)	50	(2)
10^5	40	(1.5)	50	(2)
10^6	50	(2)	75	(3)
10^7	50	(2)	100	(4)
$>10^7$	50	(2)	130	(5)

Asphalt concrete or surface treatments may be used over Type I emulsified asphalt base courses.

Asphalt concrete, or Type I emulsified asphalt with a surface treatment, may be used over Type II or Type III emulsified asphalt base courses.

The *Thickness Design* Manual, MS-1, Asphalt Institute, contains specific material requirements for all types of surface and base courses included in the design charts. Only the material requirements for the design examples below are included in this publication.

Example 1: Full-Depth Asphalt Concrete

Assume the following conditions:

Design Subgrade Modulus: $M_r = 41.37$ MPa (6000 psi)

Design Traffic: EAL $= 10^6$

From Chart 1 (Chart 4) find Full-Depth asphalt concrete design thickness equal to 240 mm (9.5 in.) asphalt concrete surface and base. From Table 11.6, the minimum surface is found to be 50 mm (2 in.), leaving the balance of 190 mm (7.5 in.) as base thickness. It is necessary that both layers meet the following requirements:

1. The aggregate gradation must comply with that contained in *Model Construction Specifications for Asphalt Concrete and Other Plant-Mix Types,* SS-1, Asphalt Institute. Tests are to be in accordance with procedures in *Mix-Design Methods for Asphalt Concrete and Other Hot-Mix Types,* MS-2, Asphalt Institute. Gradations specified by local public agencies may be used if they have a history of satisfactory performance.

2. The mixture of asphalt binder and aggregate must be designed to meet the criteria suggested in SS-1.

3. The asphalt concrete must be compacted to the specified density requirements. Refer to "Acceptance Requirements" in Specification PM-1, *Model Construction Specifications for Asphalt Concrete and Other Plant-Mix Types,* SS-1, Asphalt Institute, for directions on determining target density and relative density. Also, see Chapter 7.

Example 2: Emulsified Asphalt Mix, Type II

Assume the following conditions:

Subgrade Modulus: $M_r = 27.6$ MPa (4000 psi)
Design Traffic: EAL = 10^5

From Chart 2 (Chart 5), the total thickness required is 195 mm (7.7 in.). Table 11.7 indicates that a minimum thickness of 50 mm (2 in.) of asphalt concrete, or Type I emulsified asphalt with a surface treatment, must be placed over the emulsified asphalt base. Subtracting this from the total thickness leaves 145 mm say 150 mm (6.0 in.) for the Type II emulsified asphalt base thickness.

The aggregate and emulsified asphalt used in this mix must meet the requirements in Chapter VII, *A Basic Asphalt Emulsion Manual,* MS-19, Asphalt Institute. Design of the emulsified asphalt mixes should be accompanied by using the procedures and criteria included in MS-19.

Example 3: 150mm (6 in.) Untreated Aggregate Base

Assume the following conditions:

Design Subgrade Modulus: $M_r = 48.3$ MPa (7000 psi)
Design Traffic: EAL = 10^6

From Chart 3 (Chart 6), the design thickness of asphalt concrete over untreated aggregate base is 210 mm (8.3 in.). From Table 11.6, the minimum asphalt concrete surface course for EAL = 10^6 is 50 mm (2.0 in.). Subtracting this from the total asphalt concrete design thickness results in 160 mm, say 165 mm (6.5 in.) of asphalt concrete base mixture. The entire pavement then consists of:

50 mm (2.0 in.) asphalt concrete surface mixture,
165 mm (6.5 in.) asphalt concrete base mixture, and
150 mm (6.0 in.) untreated aggregate base.

The asphalt concrete surface and base mixtures must meet the requirements listed in Example 1 above. The untreated aggregate base must meet the following test requirements:

CBR, minimum*	80
or	
R-value, minimum*	78
Liquid Limit, maximum	25
Plasticity Index, maximum or	NP
Sand Equivalent, minimum	35
Passing 75 μm (No. 200) sieve, maximum	7

*The formulas previously stated in this lesson relating CBR and R-value to subgrade soil resilient modulus do not apply to untreated aggregate base and subbase.

11.4 Traffic Analysis and Thickness Design - Airfields

Aircraft Analysis

Types of Airports

The U.S. airport network is divided into three systems—the national system of airports, general aviation airports (including the reliever airport system) and military airports. The national system is divided into distinct subsystems, differentiated by the number of enplaned passengers.

The MS-11 manual listed in the bibliography, discusses only airports served by air carrier aircraft of greater than 270 kN (60,000 lb.) total mass (gross weight). General aviation airports are covered in *Thickness Design—Asphalt Pavements for General Aviation,* Information Series No. 154 (IS-154), Asphalt Institute. The military type is excluded, as features of these airports are specified in directives creating them.

Estimating Expected Traffic

Within any airport there exists a certain pavement area that receives more traffic than any other. This area is subjected to more aircraft movements, greater aircraft weights at departure, slower aircraft speeds, and a greater degree of traffic channelization. Each of these factors has a complex but direct bearing on the pavement thickness requirements. This area, then, should be selected as the design location for determining the maximum thickness of pavement for the airport. Invariably, the design location for an air carrier airport will be on a taxiway. Consequently, the design procedure described here considers the critical design location as the taxiway with the largest number of departures and arrivals, expressed as a percentage of the total operations for the airport.

At airports with multiple runways there usually is one runway (and its taxiways) that serves more traffic than any of the others. For these airfields, the actual total number of aircraft movements and traffic breakdown by type of aircraft can usually be obtained from statistics kept by the airport manager. Using the "wind rose" chart established at each airport, which provides a percentage breakdown of prevailing wind conditions (aircraft are usually directed to land into the wind, if possible) the specific mix or division of traffic using each of the runways and their connecting taxiways can be determined. In these cases, it will be more economical to make separate pavement thickness designs for each runway-taxiway system.

Equivalent Number of Standard Aircraft

Strain repetitions imposed on the pavement are a function of the type of aircraft, the gear load, the number of aircraft passes, and the lateral, or transverse wander characteristics of the aircraft on the design area. The optimum or design thickness is the thickness that will allow the pavement to serve, under the strains imposed by the anticipated traffic, for its design period.

Effects of strain repetitions from a mixture of different aircraft are considered cumulative. The traffic analysis procedure permits calculating the cumulative effects of traffic at different lateral intervals, X, from the taxiway centerline. This is accomplished by equating the number of strain repetitions produced by the aircraft traffic mixture to a number of equivalent strain repetitions produced by an arbitrarily selected "standard" aircraft. The "standard" aircraft is a 1590 kN (358,000 lb.) DC-8-73. The "standard" aircraft will be indicated throughout this section by the symbol

✈

This type of "standardization" procedure is analogous to the use of 80 kN (18,000 lb.) single-axle load used in highway pavement design.

Equivalency factors for several major present and proposed aircraft types have been developed and are presented in graphical form in MS-11 for easy use in the traffic analysis. The equivalency factors are functions of the aircraft type, pavement thickness, and performance criteria (ε_t, ε_c). The effect of wander of the aircraft along the taxiway is directly accounted for in the equivalency factors assuming a standard deviation of 1.1 metres (3.5 ft.).

Concept of Design

As stated by Federal Aviation Administration (FAA) Circular 150/5320-6, stabilized base and subbase courses are necessary for new pavements designed to accommodate aircraft weighing 445 kN (100,000 lb.) or more. Exceptions to this policy include only superior crushed, hard, non-frost-susceptible stone (with a minimum soaked CBR strength of 80 for base and 35 for subbase) or materials with a proven history of satisfactory performance under aircraft loadings and climatic conditions comparable to those anticipated. Therefore, Full-Depth asphalt pavement design is the preferred method for the majority of aircraft considered in this section.

The design method detailed in this section is based on the theory that a Full-Depth asphalt pavement is a multi-layered elastic system. Therefore, the application of a load to the pavement produces two critical elastic strains. They are horizontal tensile strain, ε_t, at the bottom of the asphalt concrete layer, and vertical compressive strain, ε_c, at the top of the subgrade layer. Each must be examined separately in the design analysis. The location and direction of these strains are shown in Figure 11.11.

For thickness design, each of the two types of strain, (ε_t, and ε_c) is evaluated with respect to its influence, within given limits, on the pavement's behavior. But designing to limit the two types of strain may lead to two different thickness requirements. Therefore, the final design thickness is selected as the greater of the thicknesses.

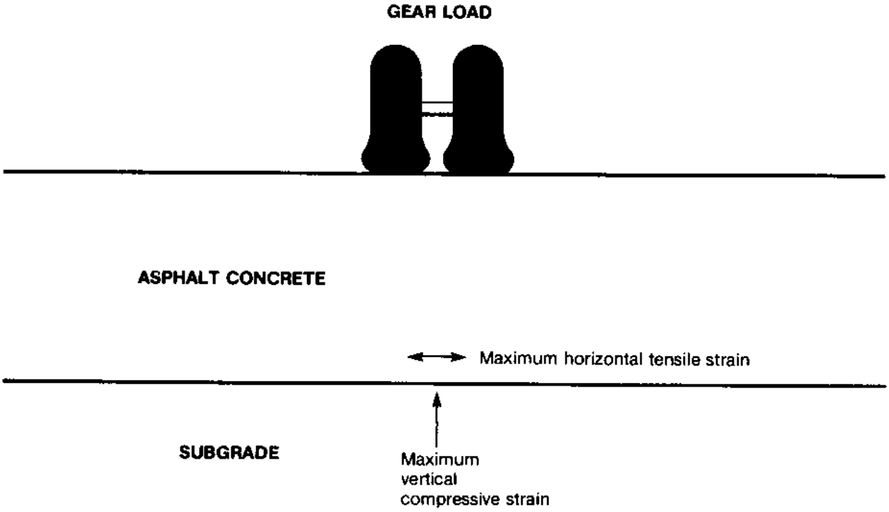

GEAR LOAD
ASPHALT CONCRETE
Maximum horizontal tensile strain
SUBGRADE
Maximum vertical compressive strain

Figure 11.11 **Location and Direction of Tensile and Compressive Strains in a Full-Depth Asphalt Pavement System.**

Materials Requirements

For information on the recommended or FAA required material specifications and compaction requirements, the reader should refer to the individual publications, MS-11 for air carrier airports and IS-154 for general aviation airports.

Thickness Design

Thickness Determination

Determining the design thickness for an airport pavement requires the following input:

- Subgrade resilient modulus
- Mean annual air temperature
- Projected forecast of aircraft traffic mix

Specific input values are used in three major steps for each strain criterion analyzed:

- Determine the Allowable Traffic Value, N_a. This yields the number of equivalent ✈ strain repetitions that a Full-Depth asphalt concrete pavement structure of a given thickness can withstand for the specific subgrade modulus and environmental combination existing at the design site.
- Determine the Predicted Traffic Value, N_p. This yields the number of equivalent ✈ strain repetitions (based on aircraft traffic forecasts) that *actually* will occur during the selected design period.

- Determine the Full-Depth asphalt concrete pavement thickness, T_A, needed to satisfy the strain criteria for the design input values of subgrade modulus, mean annual air temperature, and traffic mix. This thickness is determined by a simultaneous graphical solution of the Allowable Traffic Value, N_a, and the Predicted Traffic Value, N_p.

Figure 11.12 illustrates schematically the three steps used to determine the design thickness. In the illustration the design thickness (T_A) is based on asphalt concrete tensile strain, as it is the greater of the two thicknesses.

The design method also is available in the form of a computer program (CP-3) from the Asphalt Institute.

Example

Given the following conditions:

Mean annual air temperature: MAAT = 16° C (60° F)
Design Subgrade Modulus: M_r = 48 MPa (7000 psi)
Design Traffic: 149,000 movements of a B757-200

From Figures 11.13 and 11.14, the allowable traffic (N_a) versus thickness curve for ε_c, and ε_t, can be obtained for five levels of traffic N = 100; 1,000; 10,000; 100,000 and 1,000,000.

Using Charts A (Figure 11.15) and B (Figure 11.16) for the ε_c compressive strain criteria and Charts C (Figure 11.17) and D (Figure 11.18) for the ε_t, the predicted number of equivalent strain repetitions (N_p) can be read on the log scale, for each thickness and distance from taxiway centerline, ×. These N_p values are compiled in Figures 11.19 and 11.20 for ε_c and ε_t, respectively. In Figures 11.21 and 11.22, the two curves (N_a and N_p) are plotted showing the graphical intersection for each of the design criteria. The design thickness is the greater of the two thicknesses (intersections), rounded to the nearest 10 mm (0.5 in.).

Subgrade strain, ε_c = 419 mm (16.5 in.)
Asphalt Concrete strain, ε_t = 318 mm (15.0 in.)
Design thickness, T_A (rounded) = 420 mm (16.5 in.)

Reduced thicknesses for pavement areas other than the critical design location can be obtained from Figure 11.23.

General Aviation Design

Design thicknesses contained in the general aviation design charts are based on main gear loads equal to 47.5 percent of the gross weight. Above a gross weight of 133 kN (30,000 lb.), equivalent single wheel loads were determined for dual-tired gears. A minimum total thickness of 100 mm (4 in.) of asphalt structure is recommended for these pavements regardless of subgrade strength.

Thicknesses for Full-Depth asphalt concrete pavements for airports serving general aviation aircraft up to 270 kN (60,000 lb.) maximum gross weight can be determined with the thickness design chart in Figure 11.24.

Similarly, Figure 11.25 is a thickness design chart for emulsified bases with an asphalt concrete surface course of 50 mm (2 in.) minimum thickness.

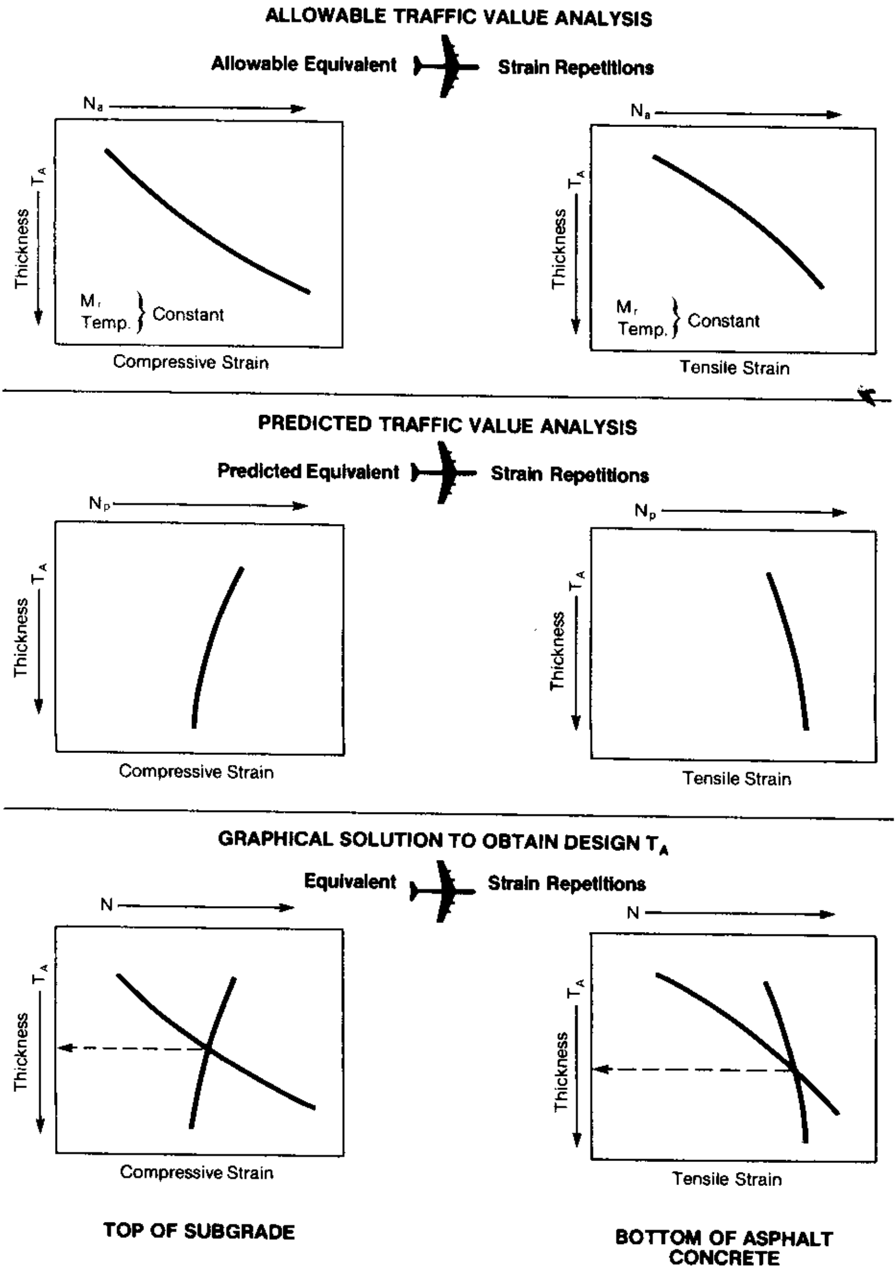


Figure 11.12 Steps Used to Determine Design Thickness.

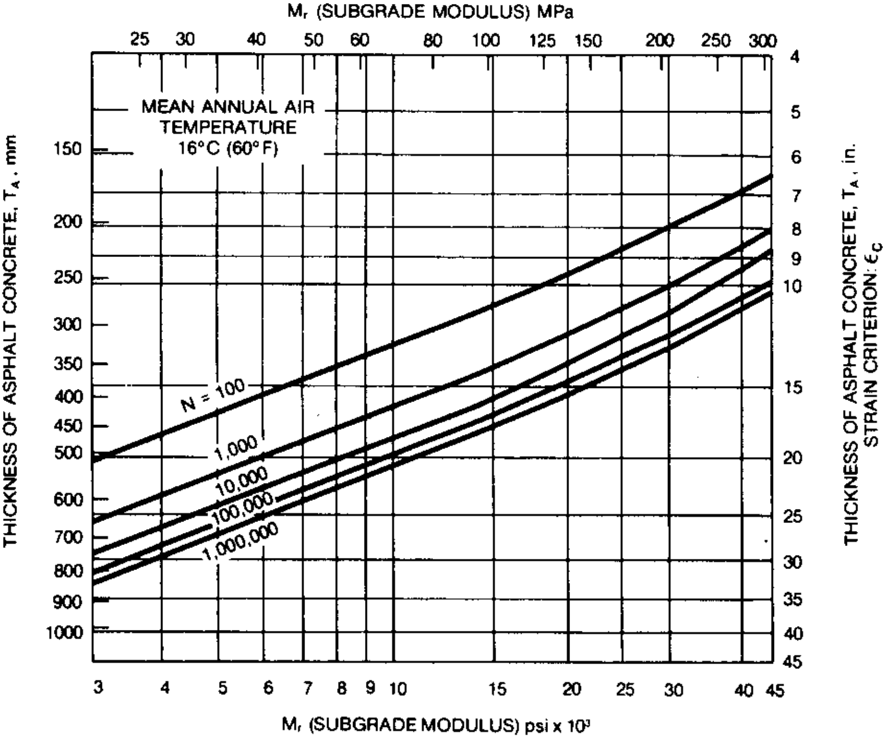


Figure 11.13 **Pavement Thickness to Limit Subgrade Vertical Compressive Strain, ε_c, Under ✈ Load Repetitions.**

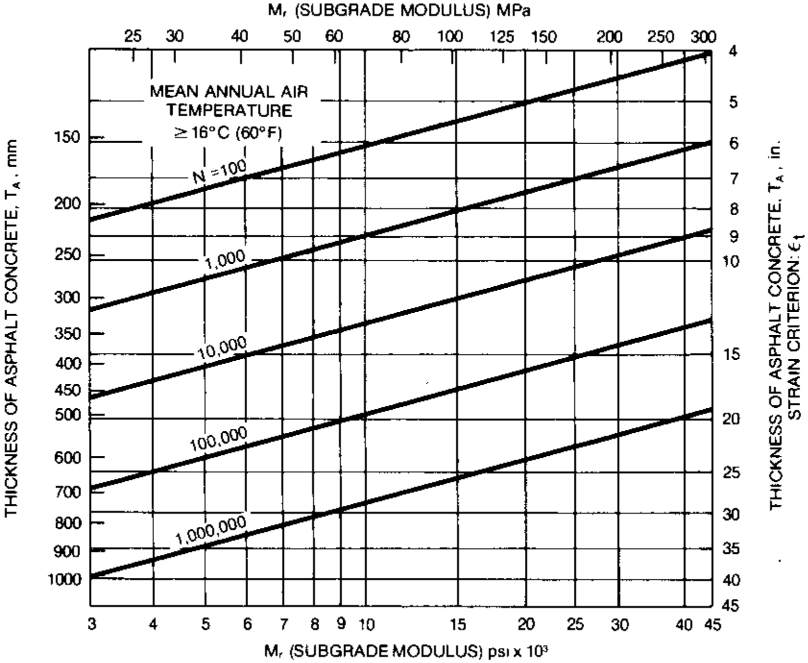


Figure 11.14 **Pavement Thickness to Limit Asphalt Concrete Horizontal Tensile Strain, ε_t, Under ✈ Load Repetitions.**

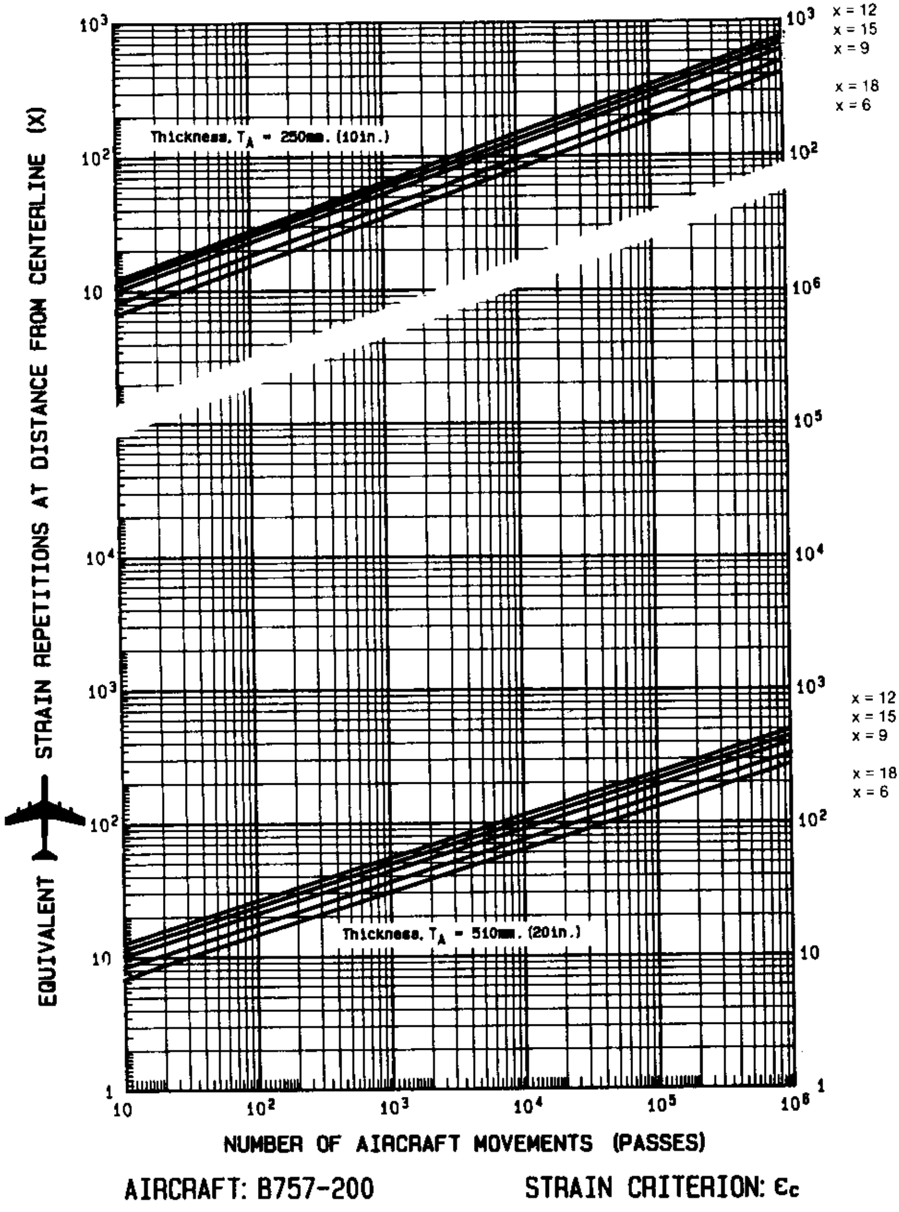


Figure 11.15 Chart A.

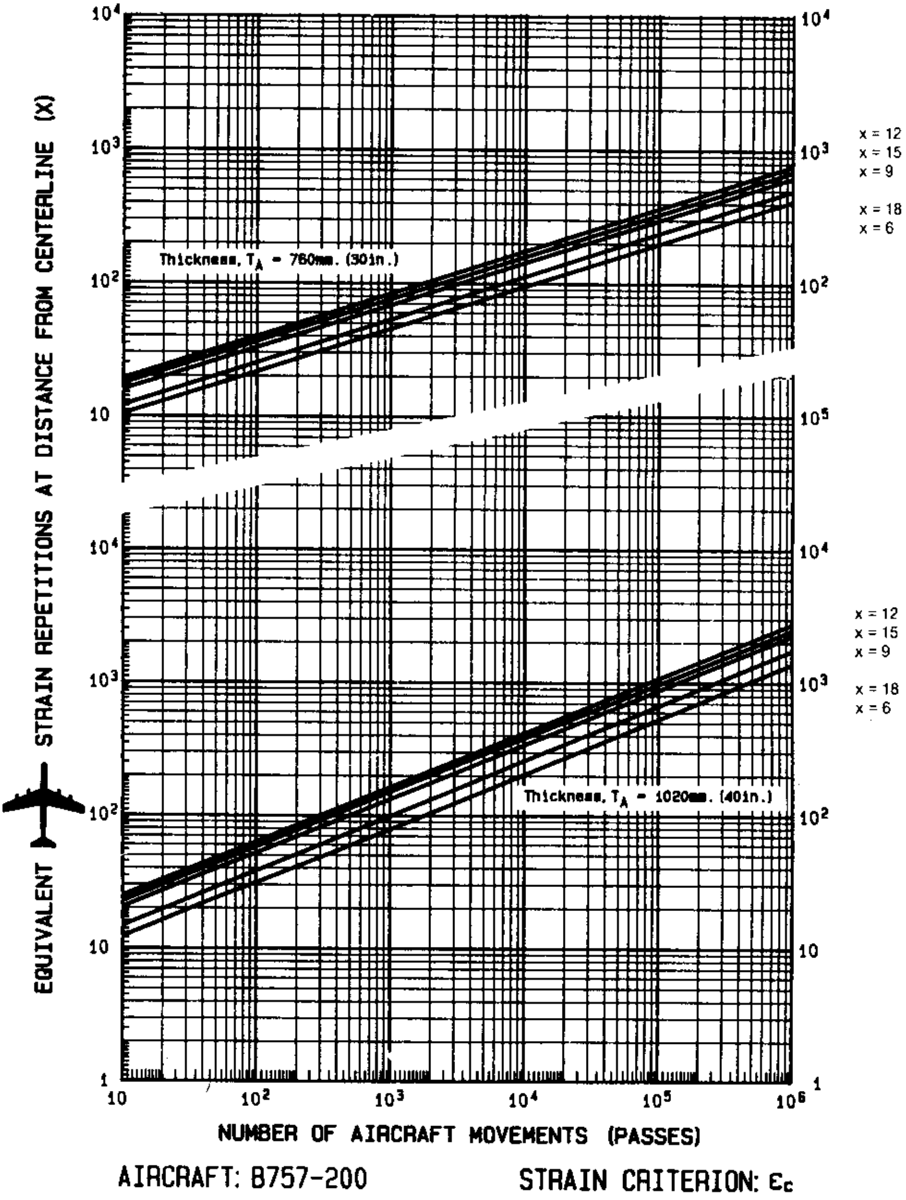


Figure 11.16 Chart B.

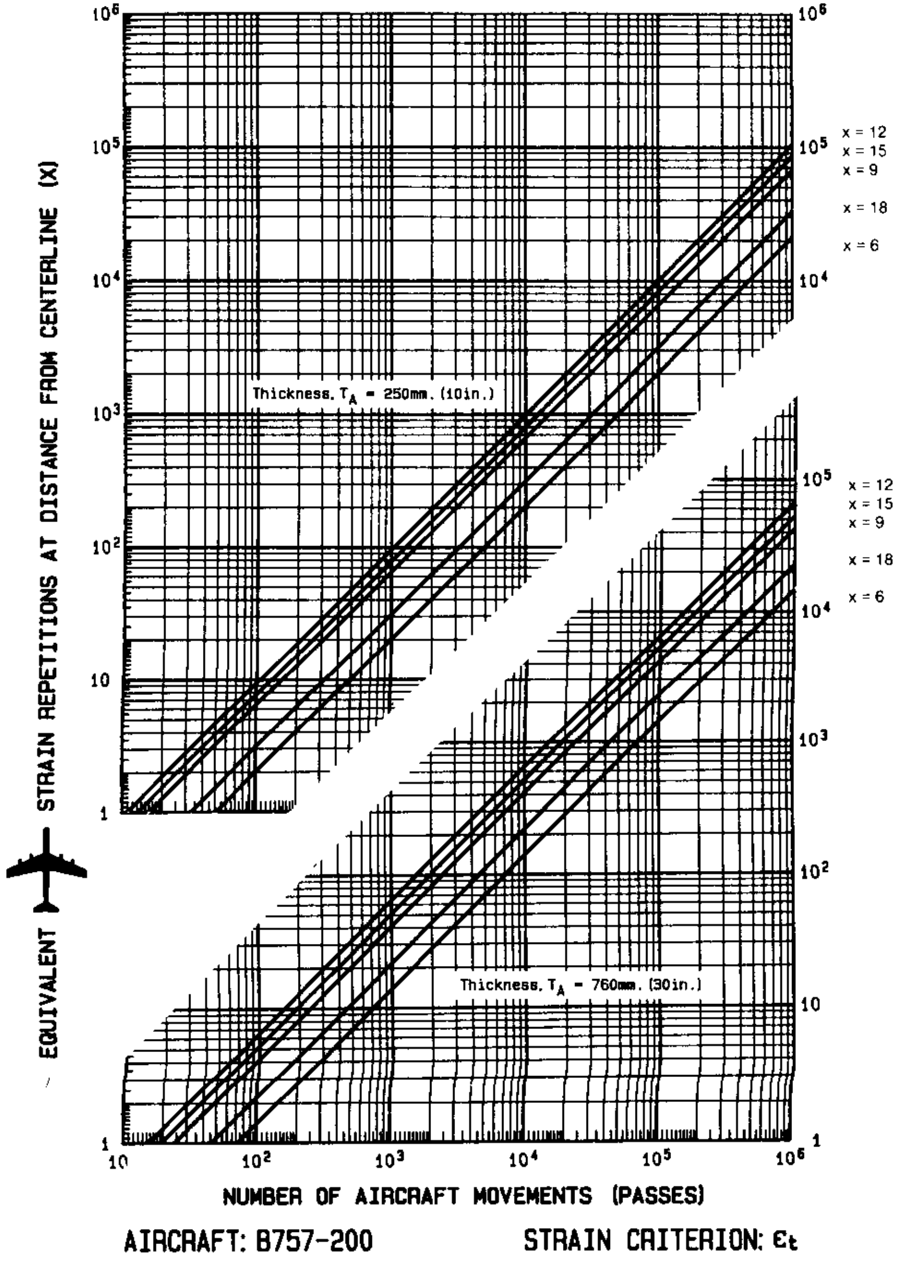


Figure 11.17 Chart C.

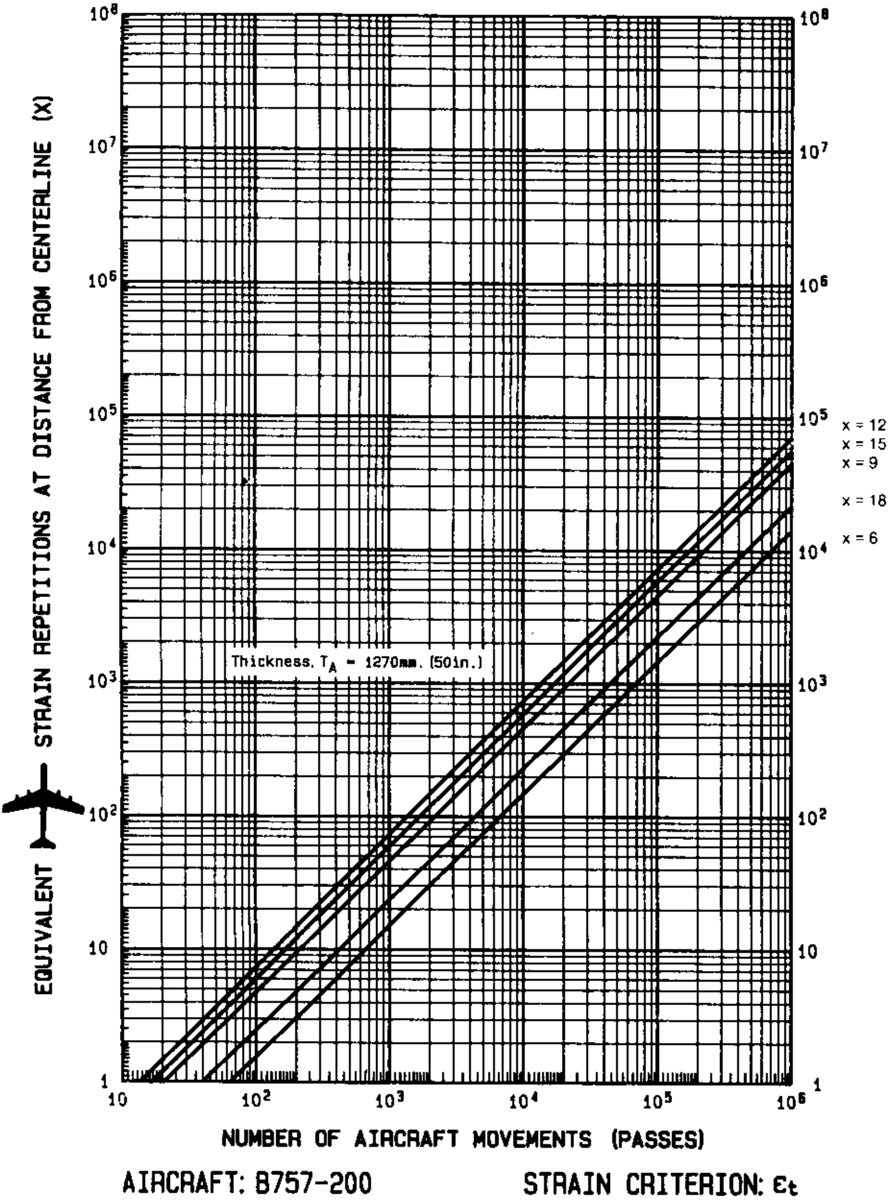


Figure 11.18 Chart D.

NUMBER OF EQUIVALENT STRAIN REPETITIONS

TYPE OF STRAIN: ε_c

DESIGN PERIOD: 20 YEARS

AIRCRAFT	MOVEMENTS IN DESIGN PERIOD	DISTANCE FROM CENTERLINE, x				
		1.8m (6 ft)	2.7m (9 ft)	3.7m (12 ft)	4.6m (15 ft)	5.5m (18 ft)
THICKNESS, T_A = 250 mm (10 in.)						
1. B 757-200	149,000	210	330	390	350	250
2.						
3.						
4.						
5.						
6.						
7.						
8.						
9.						
10.						
SUM		210	330	390	350	250
THICKNESS, T_A = 510 mm (20 in.)						
1. B 757-200	149,000	160	240	270	250	180
2.						
3.						
4.						
5.						
6.						
7.						
8.						
9.						
10.						
SUM		160	240	270	250	180
THICKNESS, T_A = 760 mm (30 in.)						
1. B 757-200	149,000	230	340	390	350	260
2.						
3.						
4.						
5.						
6.						
7.						
8.						
9.						
10.						
SUM		230	340	390	350	260
THICKNESS, T_A = 1020 mm (40 in.)						
1. B 757-200	149,000	620	1,100	1,300	1,200	800
2.						
3.						
4.						
5.						
6.						
7.						
8.						
9.						
10.						
SUM		620	1,100	1,300	1,200	800

NOTE: Circle the maximum sum for each thickness. Each circled number and its corresponding thickness is used to plot one point of the Actual Traffic Value Curve.

Figure 11.19 Aircraft Traffic Worksheet No. 2 Subgrade Vertical Compressive Strain, ε_c.

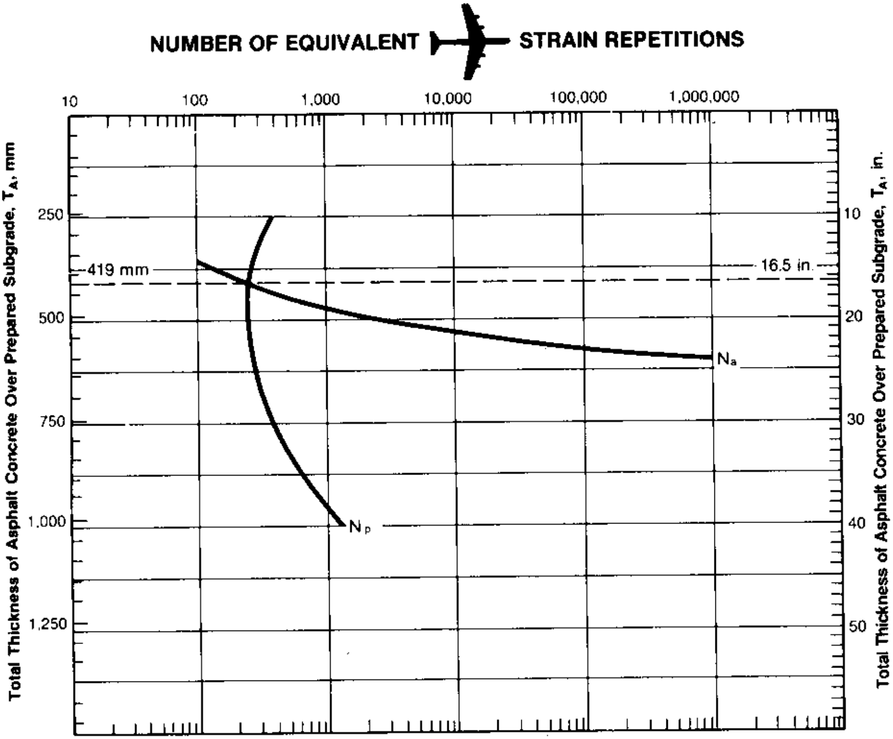
NUMBER OF EQUIVALENT STRAIN REPETITIONS

TYPE OF STRAIN: $\mathcal{E}_t$ DESIGN PERIOD: 20 YEARS

AIRCRAFT	MOVEMENTS IN DESIGN PERIOD	DISTANCE FROM CENTERLINE, x				
		1.8m (6 ft)	2.7m (9 ft)	3.7m (12 ft)	4.6m (15 ft)	5.5m (18 ft)
THICKNESS, T_A = 250 mm (10 in.)						
1. B757-200	149,000	3,100	9,900	15,000	12,000	4,900
2.						
3.						
4.						
5.						
6.						
7.						
8.						
9.						
10.						
SUM		3,100	9,900	15,000	12,000	4,900
THICKNESS, T_A = 760 mm (30 in.)						
1. B757-200	149,000	2,100	6,400	9,900	7,900	3,300
2.						
3.						
4.						
5.						
6.						
7.						
8.						
9.						
10.						
SUM		2,100	6,400	9,900	7,900	3,300
THICKNESS, T_A = 1270 mm (50 in.)						
1. B757-200	149,000	2,200	6,500	10,000	8,000	3,400
2.						
3.						
4.						
5.						
6.						
7.						
8.						
9.						
10.						
SUM		2,200	6,500	10,000	8,000	3,400
THICKNESS, T_A = mm (in.)						
1.						
2.						
3.						
4.						
5.						
6.						
7.						
8.						
9.						
10.						
SUM						

NOTE: Circle the maximum sum for each thickness. Each circled number and its corresponding thickness is used to plot one point of the Actual Traffic Value Curve.

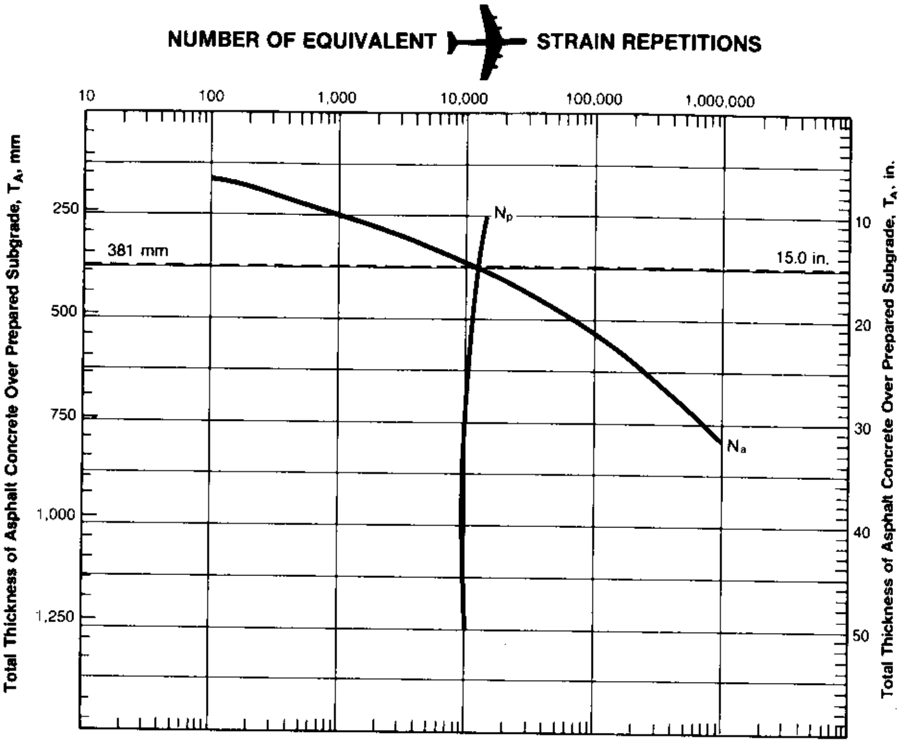
Figure 11.20 Aircraft Traffic Worksheet No. 2 Asphalt Concrete Horizontal Tensile Strain, ε_t.



M_R = 48.3 MPa (7000 psi)
temperature = 16°C (60°F)
Type of Strain = ε_c

Allowable and predicted traffic value curve worksheet

Figure 11.21 Allowable Traffic Value, N_a, and Predicted Traffic Value, N_p, Curves for Subgrade Vertical Compressive Strain, ε_c.



M_R = 48.3 MPa (7000 psi)
temperature = 16° C (60°F)
Type of Strain = ε_t

Allowable and predicted traffic value curve worksheet

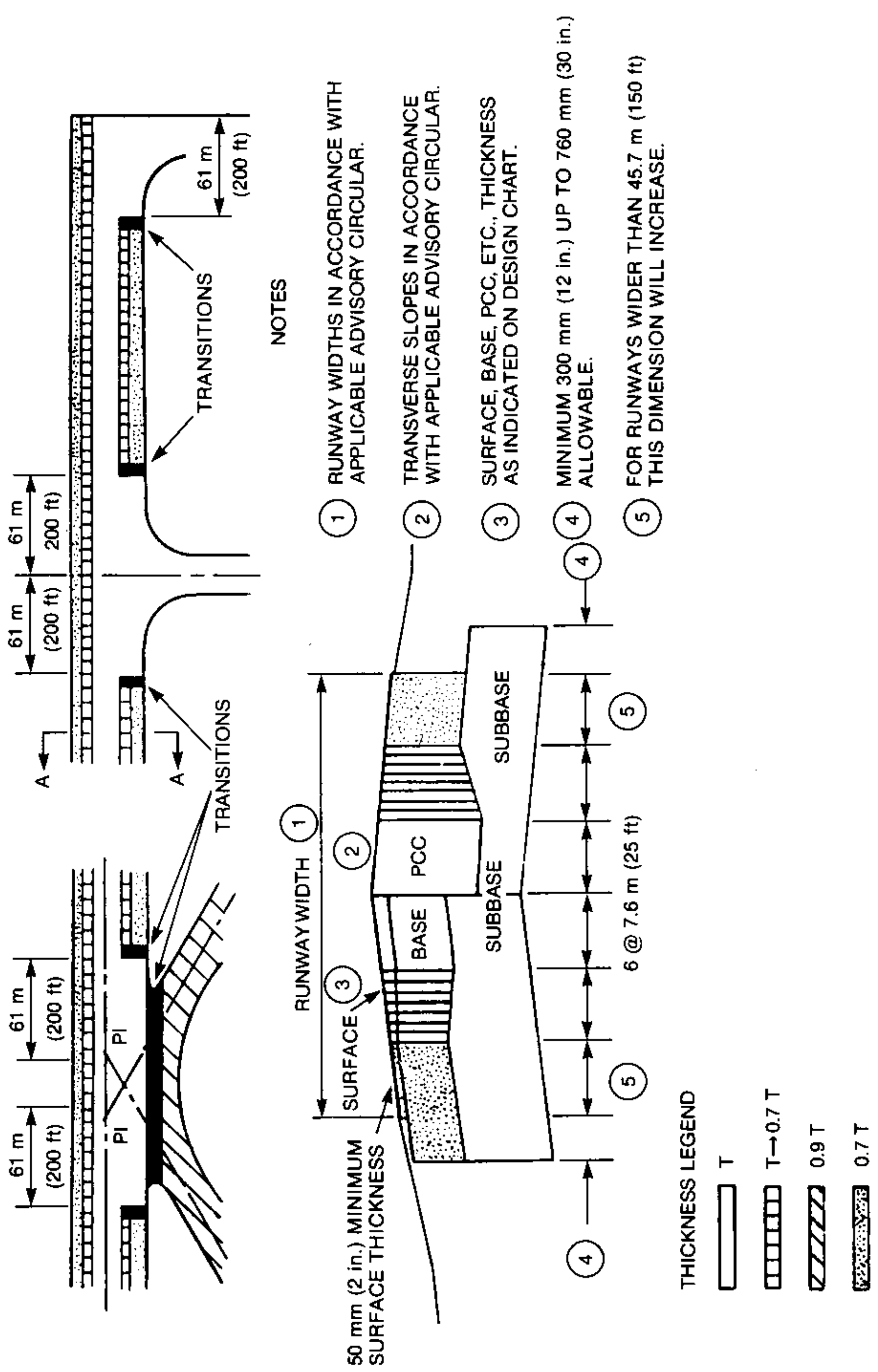
Figure 11.22 Allowable Traffic Value N_a, and Predicted Traffic Value, N_p, Curves for Asphalt Concrete Horizontal Tensile Strain, ε_t.

Also, Figure 11.26 is a design chart incorporating a 150mm (6 in.) untreated granular base course, which can be used to determine the required thickness of asphalt concrete surface above this layer.

Example—Design a general aviation runway for a mix of aircraft with the heaviest being a Beechcraft King Air at 49 kN (10,950 lb.). The soil is in a medium range, 10,500 psi (CBR = 7).

A. Full-Depth Asphalt Concrete (Figure 11.24)
Thickness of Full-Depth asphalt concrete can be read directly from the intersection of the soil value 10,500 psi (CBR = 7) and the aircraft of 49 kN (10,950 lb.):

T_A = 125 mm (5.0 in.)
(to the nearest 10 mm or ½ in.)



NOTES

① RUNWAY WIDTHS IN ACCORDANCE WITH APPLICABLE ADVISORY CIRCULAR.

② TRANSVERSE SLOPES IN ACCORDANCE WITH APPLICABLE ADVISORY CIRCULAR.

③ SURFACE, BASE, PCC, ETC., THICKNESS AS INDICATED ON DESIGN CHART.

④ MINIMUM 300 mm (12 in.) UP TO 760 mm (30 in.) ALLOWABLE.

⑤ FOR RUNWAYS WIDER THAN 45.7 m (150 ft) THIS DIMENSION WILL INCREASE.

Figure 11.23 Typical Plan and Cross-Section for Runway Pavements.

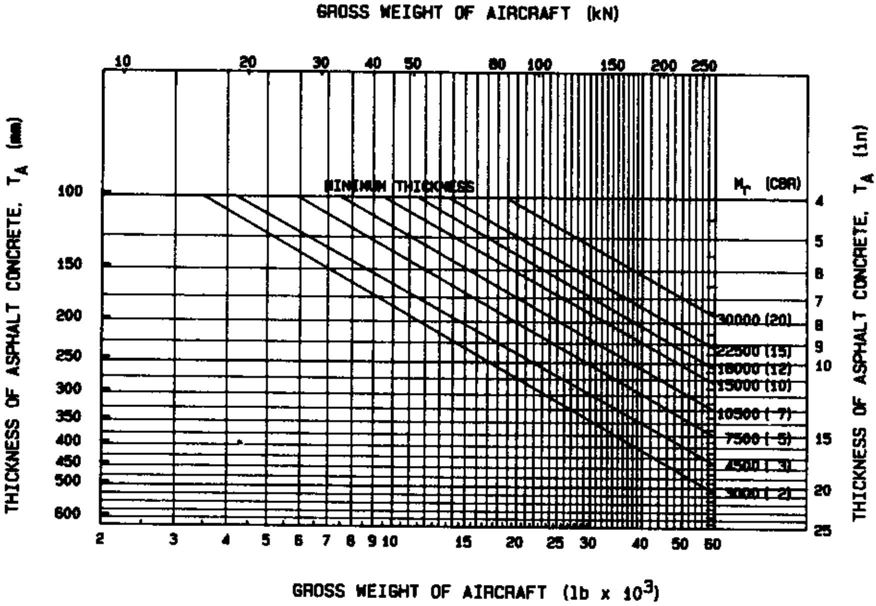


Figure 11.24 Thickness Design—Full-Depth Asphalt Concrete.

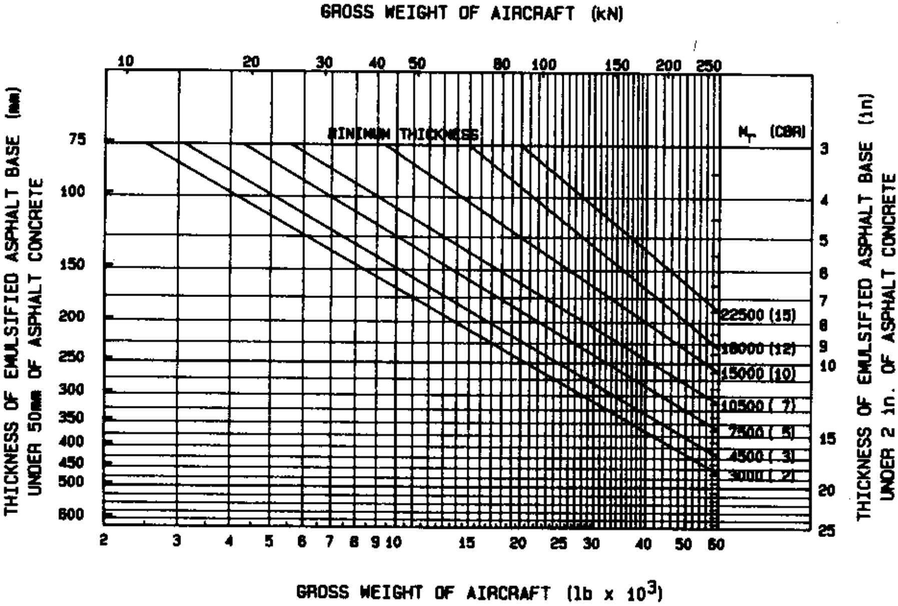


Figure 11.25 Thickness Design—Emulsified Asphalt Base Under Asphalt Concrete.

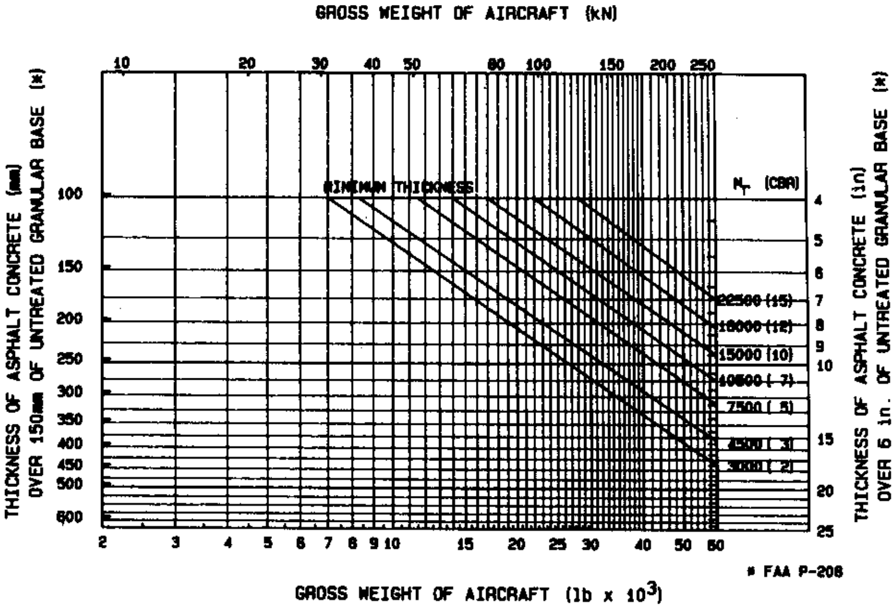


Figure 11.26 Thickness Design—Asphalt Concrete Over Untreated Base.

B. Emulsified Asphalt Base Under Asphalt Concrete (Figure 11.25)
The thickness of emulsified base is read directly from the chart (at soil value and aircraft loading):

Asphalt concrete	= 50 mm (2.0 in.)
Emulsified Asphalt Base	= 115 mm (4.5 in.)
	165 mm (6.5 in.)

C. Asphalt Concrete Over Untreated Base (Figure 11.26)
Since the intersection of the aircraft and soil value is above the minimum thickness the design is:

Asphalt concrete	= 100 mm (4 in.)
Untreated Aggregate Base	= 150 mm (6 in.)
	250 mm (10 in.)

11.5 Traffic Analysis and Thickness Design-Heavy Wheel Loads

Vehicle Analysis

Types of Vehicles

Many paved surfaces are being developed for the operation of heavy vehicles to reduce fuel and maintenance costs and to increase speed, efficiency and safety of vehicle operations. Heavy vehicles include log-hauling trucks, dump-body haulers, fork-lift trucks, straddle-carriers, rubber-tired hoists, and many special vehicles having as few as four to as many as 12 or more tires. Gross masses (weights) in excess of 1800 kN (400,000 lbs.) are being carried by some. Many of these vehicles have wheel loads and wheel-spacing characteristics that do not permit the use of standard pavement design procedures.

This section presents a procedure for designing asphalt pavements for heavy vehicles. The procedure is based on the concept that pavement design for multiple wheel vehicles can be accomplished by first calculating an "equivalent" single wheel loading. The equivalent single wheel load is then used to select pavement thicknesses. Pavement thicknesses will vary with vehicle tire-ground contact pressures, magnitude of load and spacing of the loaded tires.

Required Vehicle Data

The solution of single wheel design problems using this method requires at least the following vehicle and tire data.

P = single wheel load in kilonewtons (pounds).

p = tire-ground contact pressure in kilopascals (psi). (Preferable to tire inflation pressure).

a = radius of circular area in millimetres (inches) corresponding to a load in kilonewtons (pounds), P, with contact pressure, p, in kilopascals (psi).

The solution of dual wheel load problems requires that the spacing between tires be determined.

D = center-to-center distance between tires in millimetres (inches).

Multiple wheel problems require knowledge of the spacing between wheels and axles. The following variable is calculated from wheel spacing data, and is used in the procedure.

r = radial offset distance in millimetres (inches) from each tire to trial locations for maximum overlap of deflection.

All designs are determined assuming that the load is transmitted to the pavement on a circular loaded area. The radius (a) of this equivalent circular area is required by the design procedure. The radius of the equivalent circular area is calculated using the following equations.

If the contact area is known:

$$a = \sqrt{(\text{area})/\pi}$$

If the contact pressure is known:

$$a = \sqrt{(P/p)\pi}$$

The following example is one of five from MS-23 (see bibliography) which describes how to calculate the necessary wheel load characteristics.

Example: Log Handler

1. Determine the load to be carried.
2. Determine the equipment mass (weight) and configuration.
3. Determine the axle loads and contact pressure.

Given:

Log handler, similar to that in Figure 11.27 empty mass (weight) with ballast = 525 kN (117,950 lb.)

[Steering axle (rear) is limited to 265 kN (59,525) lb.)]

Rated payload (logs) = 353 kN (79,365 lb) at 1,370 mm (54 in.) from loaded axle

Tires:

Drive axle (front) = 21:00 × 35 (4 tires)

Steer axle (rear) = 23.5 × 25 (2 tires)

Solution:

LD = Distance from load center to front axle = 1370 mm (54 in.)

AS = Axle spacing = 7.3 metres × 1,000 = 7300 mm
(24 feet × 12 = 288 in.)

FA = Mass (weight) unloaded on front axle = 525-265 = 260 kN
(117,950 - 59,525 = 58,425 lb.)

PL = Payload = 353 kN (79,365 lb.)

Therefore:

$$\text{Front axle load} = \text{FA} + \frac{\text{PL (AS + LD)}}{\text{AS}}$$

$$= \frac{260 + 353\ (7\,300 + 1\,370)}{7300} = 679 \text{ kN}$$

$$\left[= \frac{58{,}425 + 79{,}365\ (288 + 54)}{288} = 152{,}671 \text{ lb.} \right]$$

or 679/4 = 170 kN/tire (152,671/4 = 38,168 lb./tire)

The tire service book shows for a 21:00 × 35 tire:

Gross contact area = 0.2394 m^2 (371 sq. in.)

$$\text{Ground contact pressure} = \frac{170}{0.2394} = 710 \text{ kPa}$$

$$(38{,}168/371 = 103 \text{ psi})$$

Use 710 kPa (103 psi) for design.

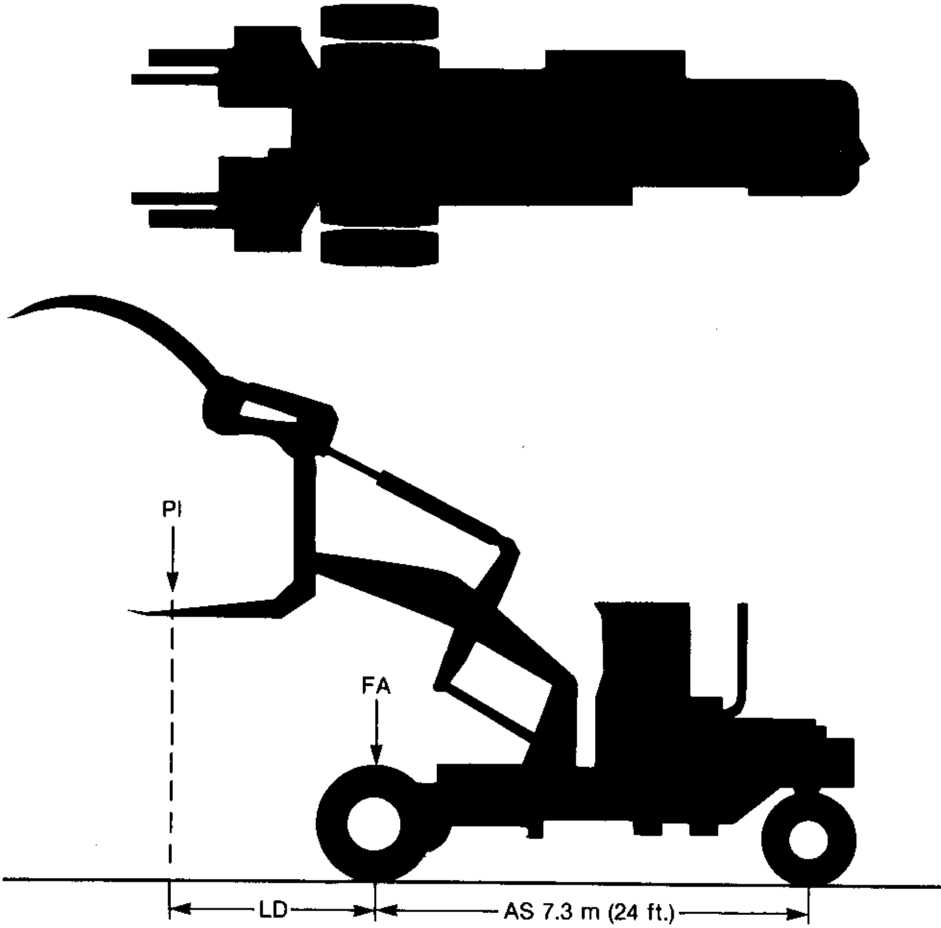


Figure 11.27 Log Handler.

This example used estimated vehicle load capacities to determine the required contact pressures. Design engineers should be aware that using 100 percent loading and channelized traffic can lead to overdesign. In many cases the use of capacity loadings may be conservative, and it may be possible to make less conservative but adequate designs using less than the rated maximum loads. Two factors are involved: the design load and the number of repetitions expected at the design load.

Studies at one port container handling facility indicate that container carriers operate empty about 50 percent of the time, and that as many as 95 percent of the containers are not fully loaded. One study revealed that use of a design load of 60 percent of the rated vehicle capacity would produce a design that would be adequate about 88 percent of the time.

The number of loaded repetitions will vary considerably, depending upon the geometric layout and other physical arrangements at the facility, the size of the load and many other characteristics. Some of the results of studies of movements of vehicles at various facilities are summarized below:

Type of Facility	**Probable No. of Repetitions for Design Per Year**
Front-end loaders	
Railroad-trailer loading and unloading on and off flat cars (approaching and/or leaving one flat car from a parallel dock)	1,650
Log handlers	12,000
Cargo container handlers	4,750
Straddle carriers (Working to painted guidelines)	26,000

Thickness Design

Concept of Design

The design procedure in this section is based upon similar multilayered elastic design concepts as used in the development of the Asphalt Institute Manuals, MS-1 and MS-11. This approach assumes that the application of a load to the pavement produces two critical elastic strains. These are horizontal tensile strain ε_t, at the bottom of the asphalt concrete layer, and vertical compressive strain, ε_c, at the top of the subgrade, as illustrated in Figure 11.11.

The design procedure in this section considers all of the above elements and was developed through an extensive analysis of theoretically computed stress-strain results. Stresses and strains are not computed directly in using the design method outlined, but the same objective is accomplished indirectly. In addition, several simplifying assumptions have been made to reduce the complexity of the design procedure.

This simplification is required since direct use of multilayered elastic analysis is a complicated design analysis. A large number of computer solutions are required to analyze the variety of loads and normal moduli ranges of asphalt concrete and subgrade soil. Additionally, the horizontal locations of maximum strains under multiple wheel loads vary with the dimensions of the load spacing, the magnitude of loads, the pavement thickness, and the modular ratios of the pavement layers.

The heavy wheel load design method treats three types of problems: single wheel load, dual wheel load and multiple wheel load problems.

Dual and multiple wheel load problems are solved by determining an equivalent single wheel load. Single wheel load design problems are reasonably simple, requiring only the magnitude of the wheel load (P), the tire-ground contact pressure (p) and the contact radius (a). Dual and multiple wheel problems are more complicated, requiring additional wheel spacing information. Whether the particular situation can be treated as a single wheel depends upon the wheel spacing of the design vehicle. If wheels are separated by a distance greater than eight times the radius of the circular loaded area (a), determined as described earlier, the problem may be treated as a single wheel design problem. However, if the distance between wheels is less than eight times the load radius, the problem should be treated as either a dual wheel load problem or a multiple load problem.

However, it may be possible to save considerable time in arriving at a solution by determining if an apparent multiple wheel load problem can be treated as either a dual wheel load problem, a succession of dual wheel load problems, or simply as a single wheel load design problem. In both dual and multiple wheel problems, the design thickness (T_A) is determined by an equivalent single wheel load calculated at the point of maximum subgrade deflection. For dual wheels this point occurs midway between the two tires. For multiple wheel load configurations, the point of maximum subgrade deflection may occur under one wheel, midway between two wheels on the rear axle, midway between all wheels on the vehicle or at some intermediate point, depending upon the dimensions and relative magnitude of loads on the individual wheels.

Allowable Thickness Design Curves

The structural design of an asphalt pavement for heavy wheel loads depends on the number and magnitude of the wheel loads to be imposed on the pavement; contact pressure; environmental conditions, such as temperature and freeze-thaw effects; subgrade strength; and pavement materials properties. For information on the recommended material requirements, the reader should refer to the MS-23 manual.

In this procedure, the allowable thickness of a Full-Depth asphalt pavement is determined in terms of p and $T_{A/a}$ where:

T_A = Full-Depth asphalt pavement thickness in millimetres (inches)
a = Radius of the equivalent single wheel load in millimetres (inches)
p = Tire-ground contact pressure in kilopascals (pound per square inch = psi)

Values of $T_{A/a}$ are selected from either Chart I (Table 11.8) or Chart II (Table 11.9), depending on the climatic region in which the facility is located.

The selection of either Chart I or Chart II is made on the basis of the following criteria:

Chart No.	Climatic Conditions (30 year averages)	Subgrade Conditions
I	Approximately 3 months with average daily temperatures 0° C (32° F) or below. Yearly average temperature of 13° C (55° F) or less.	Subgrade subject to frost weakening. Gradual increase from weak spring condition to design modulus in early fall.
II	No freezing temperatures. Yearly daily temperature above 13° C (55° F).	Subgrade modulus remains reasonably constant, throughout year, at the design value.

Once the appropriate Chart has been selected, values of $T_{A/a}$ are read directly for the values of subgrade resilient modulus (M_r), tire contact pressure (p), and the design number of load repetitions. $T_{A/a}$ is converted to T_A as follows:

- The value of tire contact radius (a) is determined as explained above. Design thickness is calculated by multiplying $T_{A/a}$ by a.
- The Asphalt Institute recommends a minimum asphalt concrete thickness, T_A, of 180 mm (7 in.) be used for heavy wheel load designs.

Table 11.8 Chart I Design $T_{A/a}$ Values for Yearly Average Daily Temperature of 13°C (55°F) or Less

Subgrade M_r (CBR)	Load Repetitions	Tire Ground Contact Pressure, kPa (psi)					
		275 (40)	415 (60)	550 (80)	690 (100)	1035 (150)	1380 (200)
30 MPa 4,500 psi (3)	10,000	0.68	0.93	1.14	1.31	1.68	2.01
	100,000	0.82	1.15	1.39	1.59	2.01	2.36
	1,000,000	1.08	1.42	1.68	1.90	2.39	2.79
50 MPa 7,500 Psi (5)	10,000	0.55	0.80	1.01	1.20	1.55	1.86
	100,000	0.74	1.02	1.25	1.43	1.88	2.23
	1,000,000	0.96	1.27	1.54	1.75	2.25	2.63
100 MPa 15,000 psi (10)	10,000	0.39*	0.59	0.81	0.98	1.33	1.62
	100,000	0.52	0.82	1.03	1.23	1.64	1.96
	1,000,000	0.74	1.06	1.32	1.54	2.00	2.36
150 MPa 22,500 psi (15)	10,000	0.39*	0.48*	0.62	0.81	1.17	1.45
	100,000	0.39*	0.63	0.88	1.08	1.47	1.81
	1,000,000	0.56	0.90	1.16	1.38	1.82	2.19

*Minimum $T_{A/a}$

Subgrade M_r (CBR)	Load Repetitions	Tire Ground Contact Pressure, kPa (psi)					
		275 (40)	415 (60)	550 (80)	690 (100)	1035 (150)	1380 (200)
30 MPa 4,500 psi (3)	10,000	0.59	0.81	1.00	1.16	1.50	1.79
	100,000	0.75	1.00	1.22	1.41	1.79	2.12
	1,000,000	0.92	1.22	1.46	1.68	2.11	2.47
50 MPa 7,500 psi (5)	10,000	0.47	0.70	0.88	1.04	1.37	1.63
	100,000	0.63	0.87	1.10	1.28	1.66	1.97
	1,000,000	0.81	1.08	1.33	1.55	1.97	2.33
100 MPa 15,000 psi (10)	10,000	0.39*	0.48*	0.67	0.84	1.17	1.44
	100,000	0.40	0.67	0.89	1.07	1.45	1.75
	1,000,000	0.60	0.88	1.13	1.35	1.75	2.10
150 MPa 22,500 psi (15)	10,000	0.39*	0.48*	0.55*	0.67	1.03	1.29
	100,000	0.39*	0.51	0.73	0.93	1.30	1.60
	1,000,000	0.42	0.74	0.98	1.20	1.59	1.93

*Minimum $T_{A/a}$

Thickness Determination for Dual Wheel Loads

The design procedure for dual wheel load problems requires conversion to an *equivalent* single wheel load that can be used with Chart I or II to determine the required thickness.

Dual wheel load problems are solved by a graphical solution of two T_A versus load relationships, an *allowable* T_A versus P relationship, discussed previously and an equivalent load (P_e) versus T_A relationship.

Example:

Determine an *allowable* single wheel load design curve for the following conditions:

P = 88 to 309 kN (19,840 to 69,450 lb.)
p = 690 kPa (100 psi)
M_r = 50 MPa (7,500 psi)
Repetitions = 10,000
$T_{A/a}$ = 1.20 from Chart I [MAAT 13° C (55° F)]

Values of equivalent single wheel load (a) and asphalt thickness T_A are shown in Figure 11.28, and plotted on Figure 11.29.

Determine the equivalent single wheel load curve for a 222 kN (50,000 lb.) dual load [P = 111 kN (25,000 lb.)], p = 690 kPa (100 psi) and supported on a subgrade with M_r = 50 MPa (7,500 psi). The center-to-center spacing is 610 mm (24 in.) for the dual wheels.

Therefore:
P = 111 kN (25,000 lb.)
p = 690 kPa (100 psi)
D = 610 mm (24 in.)

$$a = \sqrt{\frac{P}{p\pi}} = \sqrt{\frac{111}{690\pi}} = 0.226 \text{ m or } 226 \text{ mm}$$

$$\left(= \sqrt{\frac{25000}{100\pi}} = 8.9 \text{ in.} \right)$$

$D/a = 610/226 = 2.7$ ($24/8.9 = 2.7$) Since D/a is less than 8.0 the two tires must be analyzed as an equivalent dual wheel load.

P		$a = \sqrt{\frac{P}{p\pi}}$		$T_A = (T_{A/a})\ a.$	
kN	(lb)	mm	(in.)	mm	(in.)
88	(19,840)	200	(8.0)	240	(9.6)
132	(29,760)	250	(9.7)	300	(11.6)
176	(39,680)	285	(11.2)	342	(13.4)
221	(49,600)	320	(12.6)	384	(15.1)
265	(59,520)	350	(13.8)	420	(16.6)
309	(69,450)	380	(14.9)	456	(17.9)

Figure 11.28 Data for Allowable Single Wheel Load Design Curve.

For $D/a = 2.7$ Figure 11.30 was used to obtain load factors (L) that correspond to $T_{A/a}$ equal to 1.0, 1.5, 2.0, 2.5 and 3.0. Equivalent wheel load (P_e) and asphalt thickness (T_A) were calculated using the relationships $P_e = 2P/L$ and $T_A = a\ (T_{A/a})$ for each value of $T_{A/a}$ selected. The data and calculated values are shown in Figure 11.31. Values of P_e versus T_A are plotted in Figure 11.29.

For this example problem, the design asphalt thickness (T_A) is determined by the intersection of the allowable T_A versus P curve and the P_e versus T_A curve, shown in Figure 11.29, the design T_A = 370 mm (14.6 in.). This corresponds to a P_e = 204 kN (45,800 lb.).

Had there been more wheels, either single or dual, located within an 8 radii distance, the problem would have been solved using the multiple wheel procedure.

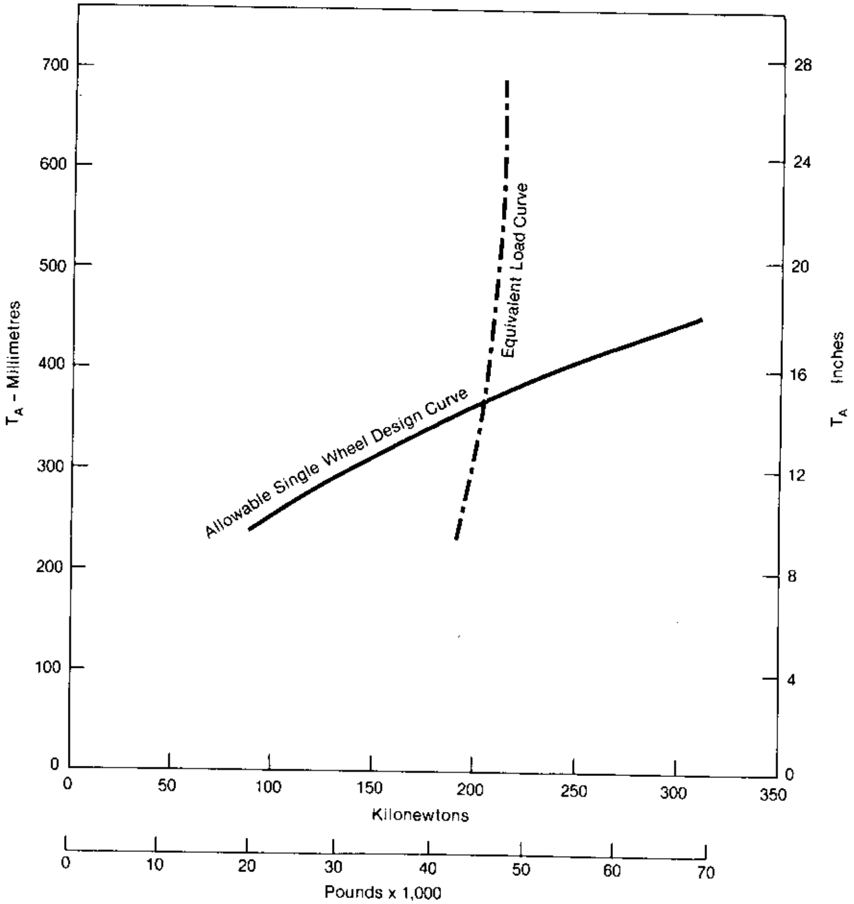


Figure 11.29 Allowable Versus Equivalent Single Wheel Curves.

Thickness Determination for Multiple Wheel Loads

The design procedure for more than two wheel loads is considerably more tedious and complicated. For this reason, the manual graphical approach cannot be presented here. The reader should refer to either MS-23 or the CP-2 computer program which is available from the Asphalt Institute. The computer program will handle up to 20 wheel loads and perform the necessary design computations of the multiple-wheel load procedure.

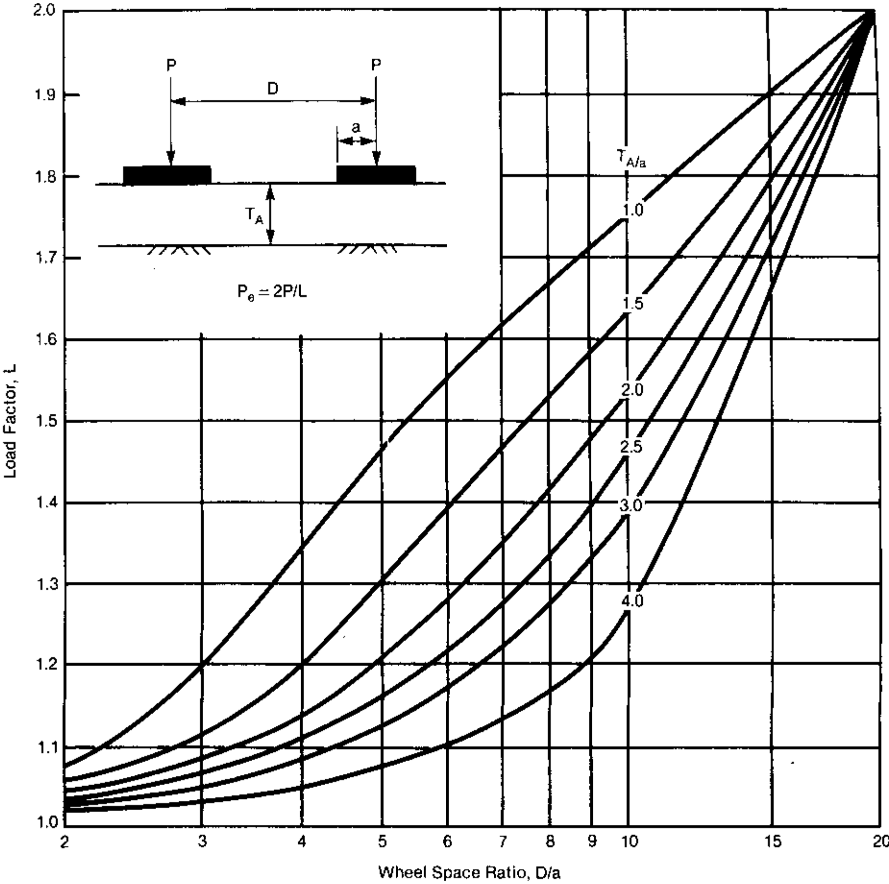


Figure 11.30 Dual Wheel Load Factors.

a		$T_{A/a}$	L	$P_e = \frac{222}{L}\left(\frac{50{,}000}{L}\right)$		T_A	
mm	(in.)			kN	(lb.)	mm	(in.)
226	(8.9)	1.0	1.16	191	(43,100)	229	(8.9)
		1.5	1.10	202	(45,500)	344	(13.4)
		2.0	1.07	207	(46,700)	458	(17.8)
		2.5	1.06	209	(47,200)	573	(22.3)
		3.0	1.04	213	(48,100)	687	(26.7)

Figure 11.31 Data for Equivalent Single Wheel Load Determination.

11.6 Planned Stage Construction

Advantages

The planned stage construction of pavements is accomplished by constructing part of the required pavement thickness and applying successive layers according to a predetermined time schedule. Planned stage construction should not be confused with major maintenance or rehabilitation of existing pavements. The procedure is based on the presumption that the second stage will be constructed before the first stage shows serious signs of distress.

Stage construction has several advantages:

1. When there is not enough money to construct the full design thickness, the pavement may be designed for construction in two stages—the first stage being designed for a shorter time period. However, it is important that plans be made to have funds available when the second stage is to be constructed.
2. Difficulties in estimating traffic for periods of 20 to 25 years, particularly for airports and highways in rural locations scheduled for development, make planned stage construction attractive. A facility can be designed for a shorter time period and traffic counts can then be made when the facility is in use. From this in-service data, improved estimates of future traffic are possible.
3. Experience indicates that pavements overlaid after being subjected to traffic perform somewhat better than new pavements of equal design.
4. Weak spots that develop during the first stage can be repaired and revised traffic data will be available. By carefully evaluating the actual condition of the pavement near the planned end of the first stage, savings in final thickness or extended life of the original pavement may be realized.

Design Approach

Design for stage construction is based on the concept of remaining life. Since the procedure involves planned stage construction, the pavement is designed on the presumption that the planned overlay will be placed before the pavement has used up all of its fatigue life.

The stage design period should equal to 25 to 50 percent of the total design period for the project. For example, if the design period is 20 years, then the stage design period should be 5 to 10 years.

To ensure that the first stage layer of asphalt concrete will function effectively with the second, the second layer must be applied when the cumulative damage in the first layer does not exceed about 60 percent. That is, the remaining life in the existing layer is at least 40 percent of its design life.

The following symbols are used in this procedure:

n_1 = Design traffic for stage 1
n_2 = Design traffic for stage 2
N_1 = adjusted design traffic for stage 1
N_2 = adjusted design traffic for stage 2
h_1 and h_2 = design thickness for stage 1 and stage 2 respectively
h_s = thickness to be added at stage 2

The steps used to obtain the stage 1 and stage 2 design thicknesses are:

1. Determine the design traffic that is predicted for the first and second stages (n_1 and n_2)
2. Calculate adjusted design traffic values
   $N_1 = (100/60\%\text{ damage}) \times n_1 = 1.67\ n_1$
   $N_2 = (100/40\%\text{ remaining life}) \times n_2 = 2.5n_2$
3. From the normal design procedure, determine the design thicknesses, h_1 and h_2, for the adjusted design traffic values, N_1 and N_2.
4. Determine the thickness to be added at stage 2
   $h_s = h_2 - h_1$

Note: When the end of the first stage time period approaches, this planned additional second stage design thickness may be altered based on the current condition and/or current strength and traffic measurement.

Example: Assume that a Full-Depth asphalt pavement will be designed for a subgrade resilient modulus 34.47 MPa (5000 psi); 200,000 EAL repetitions are expected during the first five years, and 1.8×10^6 repetitions during the following 15 years. Prepare a design for this project considering the first stage to be five years and that an overlay will be placed at that time to accommodate the additional traffic expected during the next 15 years. Follow the above steps to obtain:

1. Design EAL values:
   $n_1 = 200{,}000$; $n_2 = 1{,}800{,}000$
2. Adjusted Design EAL values:
   $N_1 = 1.67 \times 200{,}000 = 334{,}000$; $N_2 = 2.5 \times 1{,}800{,}000 = 4{,}500{,}000$
3. From Design Chart, Figure 11.5 (or Figure 11.8):
   $h_1 = 215$ mm (8.5 in.); $h_2 = 330$ mm (13.0 in.)
4. The first stage design thickness, $h_1 = 215$ mm (8.5 in.)
5. The added thickness planned for stage 2 is
   $H_s = 330 - 215 = 115$ mm ($13.0 - 8.5 = 4.5$ in.)

11.7 Pavement Overlay Design

Evaluating Structural Adequacy

Structural evaluation determines the adequacy of a pavement to support traffic without developing appreciable structural distress. The intent of structural evaluation is to determine both the current adequacy of the pavement and to predict its future service life with respect to the traffic using it. When the pavement is found to be inadequate for current or future use, the evaluation forms a basis for designing the improvements needed to provide proper service.

A pavement structure may be inadequate for several reasons. The pavement may have simply exceeded its design life—traffic volume and weight may have increased at a rate greater than anticipated in the original design. In such cases the service life is reduced. Some properties of the pavement's materials may have changed under service conditions—this could have reduced the effectiveness of the material in the pavement structure. Many existing pavements were built before the development of design procedures that take into account the relationships between subgrade support, strength of paving materials, and vehicle loadings; thus they may not comply with current structural requirements.

Evaluation techniques for pavements fall into two general categories:

1. *Deflection Method*

This method utilizes various pieces of specialized equipment to measure deflections in the existing pavement to indicate remaining service life and structural overlay requirements. Because of the many methods available, they will not be discussed herein. Detailed information on deflection methods is found in Asphalt Institute Manual MS-17 or the 1986 AASHTO Design Guide.

2. *Effective Thickness Method*

This method utilizes *visual* condition surveys to indicate the structural condition of the existing pavement. Pavement structural components are evaluated so that a representative Effective Thickness can be assigned and used in assessing current adequacy.

Effective Thickness Analysis

The Effective Thickness procedure for the structural evaluation and design of overlays is based on the concept that pavements deteriorate, i.e., have reduced life, after exposure to traffic for extended periods of time. In effect, a pavement "uses" part of its total life as a result of load repetitions imposed by traffic. By the time distress conditions appear on the surface of the pavement, a certain amount of the useful life of the pavement has been used and must be accounted for in the design process. At the same time the "remaining life" of the existing pavement can be utilized in designing the pavement for future conditions.

The procedure assumes that as a pavement uses part of its total life, it behaves as if it were an increasingly thinner pavement, i.e., its effective thickness becomes less and less to account for the expended portion of the total life of the pavement.

To calculate the effective thickness of an existing pavement and to select and design the appropriate method of rehabilitation it is necessary to know the composition of each pavement layer, the thickness of each pavement layer, and the properties of the subgrade. The Effective Thickness (T_e) of an existing pavement is the thickness that it would be if it could be converted to Full-Depth asphalt concrete. If strengthening is called for, this Effective Thickness also can be used in designing the additional required pavement thickness. The existing pavement structure, then, will constitute an integral and major part of the redesigned pavement. To determine T_e, each layer of the existing pavement must be converted to equivalent thickness of asphalt concrete using the proper Conversion Factor (Table 11.10) for each.

If original records are not available, or if necessary to obtain complete information, granular layers should be sampled and tested to determine whether they classify as granular base, subbase, or improved subgrade and to determine layer thicknesses. Layers originally classed as granular base, for example, may actually be no better than subbase or improved subgrade because of the intrusion of soil and the degradation of aggregate.

Asphalt layers should be examined to determine the types of mixture and the condition and thickness of each.

Portland cement concrete should be examined to determine slab condition, thickness, and support. Among the signals of distress to look for are pumping, cracking, spalling, faulting of joints, and slab movement under traffic.

Evaluating the condition of asphalt or portland cement concrete layers is largely a judgmental determination and its classification, to a considerable degree, depends upon the experience of the observer.

Once component layers are identified and their condition determined, appropriate Conversion Factors are selected from Table 11.10. The Effective Thickness (T_e) of each layer is computed as the product of the measured layer thickness and the Conversion Factor. Effective thickness of the total pavement structure is the sum of each layers Effective Thickness.

Overlay Thickness Determination

With the subgrade strength value, traffic analysis, and Effective Thickness (T_e) derived from the pavement examination, one can determine the thickness of overlay needed to strengthen an inadequate pavement so it will

Table 11.10 **Conversion Factor for Converting Thickness of Existing Pavement Components to Effective Thickness (T_e)**

(These conversion factors apply ONLY to pavement evaluation for overlay design. In no case are they applicable to original thickness design.)

Classification of Material	Description of Material	Conversion Factors*
I	a) Native subgrade in all cases	0.0
	b) Improved Subgrade**—predominantly granular materials—may contain some silt and clay but have P.I. of 10 or less	
	c) Lime modified subgrade constructed from high plasticity soils—P.I. greater than 10.	
II	Granular Subbase or Base—Reasonably well-graded, hard aggregates with some plastic fines and CBR not less than 20. Use upper part of range if P.I. is 6 or less; lower part of range if P.I. is more than 6.	0.1-0.2
III	Cement or lime-fly ash stabilized subbases and bases** constructed from low plasticity soils—P.I. of 10 or less.	0.2-0.3
IV	a) Emulsified or cutback asphalt surfaces and bases that show extensive cracking, considerable raveling or aggregate degradation, appreciable deformation in the wheel paths, and lack of stability.	0.3-0.5
	b) Portland cement concrete pavements, (including those under asphalt surfaces) that have been broken into small pieces 0.6 metre (2 ft) or less in maximum dimension, prior to overlay construction. Use upper part of range when subbase is present; lower part of range when slab is on subgrade.	
	c) Cement or lime-fly ash stabilized bases** that have developed pattern cracking, as shown by reflected surface cracks. Use upper part of range when cracks are narrow and tight; lower part of range with wide cracks, pumping or evidence of instability.	

*Values and ranges of Conversion Factors are multiplying factors for conversion of thickness of existing structural layers to equivalent thickness of asphalt concrete.

**Originally meeting minimum strengths and compaction requirements specified by most state highway departments

Classification of Material	Description of Material	Conversion Factors*
V	a) Asphalt concrete surface and base that exhibit appreciable cracking and crack patterns.	0.5-0.7
	b) Emulsified or cutback asphalt surface and bases that exhibit some fine cracking, some raveling or aggregate degradation, and slight deformation in the wheel paths but remain stable.	
	c) Appreciably cracked and faulted portland cement concrete pavement (including such under asphalt surfaces) that cannot be effectively undersealed. Slab fragments, ranging in size from approximately one to four square metres (yards), and have been well-seated on the subgrade by heavy pneumatic-tired rolling.	
VI	a) Asphalt concrete surfaces and bases that exhibit some fine cracking, have small intermittent cracking patterns and slight deformation in the wheel paths but remain stable.	0.7-0.9
	b) Emulsified or cutback asphalt surface and bases that are stable, generally uncracked, show no bleeding, and exhibit little deformation in the wheel paths.	
	c) Portland cement concrete pavements (including such under asphalt surfaces) that are stable and undersealed, have some cracking but contain no pieces smaller than about one square metre (yard).	
VII	a) Asphalt concrete, including asphalt concrete base, generally uncracked, and with little deformation in the wheel paths.	0.9-1.0
	b) Portland cement concrete pavement that is stable, undersealed and generally uncracked.	
	c) Portland cement concrete base, under asphalt surface, that is stable, non-pumping and exhibits little reflected surface cracking.	

handle anticipated traffic for some projected time. To find the thickness of overlay needed:

1. Determine the Design Subgrade Resilient Modulus, M_r. (Refer to previous section.)
2. Determine the estimated future design traffic.
3. Using the design chart for the appropriate environmental condition, determine the Full-Depth asphalt concrete pavement thickness, T_A, needed to satisfy the subgrade and traffic conditions determined in Steps 1 and 2 (above) for the selected Design Period.
4. Determine the Effective Thickness, T_e, of the existing pavement structure as detailed above.
5. The thickness of asphalt concrete overlay required, then, is equal to T_A-T_e.

When placing an overlay over portland cement concrete pavements, there is always a concern for reflection cracking. The design thickness computed in Step 5 should always be checked for this additional concern.

If the structural evaluation indicates that reflection cracking may be the controlling factor in performance, the overlay thickness design, discussed above, may need to be modified. As described in Chapter 9 and more thoroughly in *Asphalt Overlays for Highway and Street Rehabilitation,* Manual Series No. 17 (MS-17), Asphalt Institute, there are a number of methods currently showing promise for controlling reflection cracking in certain situations:

1. Cracking and seating of deteriorated pcc pavement.
2. Rubblizing the pcc pavement.
3. Incorporating an open-graded crack-relief layer in the overlay.
4. Recycling the top layer(s) of the cross-section.
5. Incorporating an asphalt-rubber interlayer in the overlay.
6. Laying a fabric interlayer with tack coat prior to overlay.
7. Sawcut and seal the overlay directly over the portland cement concrete joint.

Depending on the amount of horizontal and vertical movement occurring in the existing pavement, methods 6 and 7 may have limited effectiveness. Excessive vertical movement of pcc slabs should be initially alleviated through undersealing as detailed in the Asphalt Institute Construction Leaflet No. 13 (CL-13) *Specifications for Undersealing Portland Cement Concrete Pavements with Asphalt.*

Example

Determine the effective thickness of a three-layer pavement consisting of a 100 mm (4 in.) asphalt concrete surface, a 150 mm (6 in.) portland cement stabilized base and a 100 mm (4 in.) untreated crushed gravel base. The surface shows numerous transverse cracks and considerable alligator cracking in the wheel paths. The portland cement-stabilized base shows signs of pumping and loss of stability along the pavement edges. The conversion factors, C = 0.5 for the surface, C = 0.3 for the cement-stabilized base and C = 0.2 for the crushed gravel base, are determined from Table 11.10.

T_e (AC surface) = 100 × 0.5 = 50 mm (2.0 in.)
T_e (cement-stabilized base) = 150 × 0.3 = 45 mm (1.7 in.)
T_e (gravel base) = 100 × 0.2 = 20 mm (0.8 in.)
T_e (all layers) 115 mm (4.5 in.)

Design an overlay for the pavement described:
Given: Subgrade M_r = 82.7 MPa (12,000 psi)
Design EAL = 2×10^6 repetitions
T_e (all layers) = 115 mm (4.5 in.)
From Design Chart, Figure 11.5 (or Figure 11.8),
T_A = 240 mm (9.5 in.)
$T_o = T_A - T_e$
T_o = 240 - 115 = 125 mm (4.9 in.)
Use 130 mm (5.0 in.)

Chapter 12
Hot-Mix Recycling

Hot-mix recycling of asphalt pavements is a process that has been increasingly used during the last several years, and has become a major option in hot-mix asphalt production. Current interest in hot-mix recycling has arisen due to several circumstances. The continuing concern over the effective use of energy, the economic advantage gained through the reuse of asphalt, depletion of readily available aggregate sources, and increased production costs, all have given impetus to the use of hot-mix recycling.

Although the operation of hot-mix recycling facilities requires more specific detail than can be supplied in this Chapter, the basic principles outlined herein will provide an initial introduction.

SECTION 12.1 Hot-Mix Recycling Background
☐ Introduction
☐ Definitions
☐ Thickness Design

SECTION 12.2 Mix Design
☐ Introduction
☐ Preparatory Steps
☐ Material Evaluation
☐ Mix Design

SECTION 12.3 Reclaimed Materials
☐ Pavement Removal
☐ Pavement Size Reduction
☐ Stockpiling

SECTION 12.4 Facility Production Methods
☐ Production Options
☐ Batch Facilities
☐ Drum-Mix Facilities
☐ Spreading and Compaction

BIBLIOGRAPHY

1. *Asphalt Hot-Mix Recycling, MS-20,* Asphalt Institute.
2. *Guidelines for Recycling Pavement Materials,* National Cooperative Highway Research Program Report 224, Transportation Research Board, September 1980.

12.1 Hot-Mix Recycling Background

Introduction

Foremost among the concerns of the pavement construction industry, and funding agencies everywhere, is the conservation of materials, protection of the environment, and economy of construction/rehabilitation procedures.

Pavement recycling with asphalt is a procedure that eminently and realistically meets these concerns, especially since it is economical in terms of energy and materials.

In asphalt pavement recycling, materials reclaimed from old pavement structures are reprocessed along with some new materials to produce asphalt mixtures meeting all normal specification requirements. The recycled mixtures may be placed on the same roadbeds from which the reclaimed materials came; or they can be used anywhere that asphalt mixtures are needed.

Asphalt recycling is not a new technique; its history stretches back several decades. However, increasing economic and environmental needs have brought added emphasis and many technical refinements to recycling procedures.

Definitions

Recycling is the reuse, usually after some processing, of material that has served its first intended purpose. Recycling may encompass several forms—surface recycling, cold-mix recycling, and hot-mix recycling, but only hot-mix recycling is covered in this Chapter. Cold-mix recycling is discussed in Chapter 13. The following definitions apply to the recycling process:

1. Reclaimed asphalt pavement (RAP)—removed and/or reprocessed pavement materials containing asphalt and aggregates.
2. Reclaimed aggregate material (RAM)—removed and/or reprocessed pavement materials containing no reusable binding agent.
3. Hot-mix recycling—a process in which reclaimed asphalt pavement (RAP) materials, reclaimed aggregate materials (RAM), or both, are combined with new asphalt, and/or recycling agents, and/or new aggregate, as necessary, in a central mixing facility to produce hot-mix paving mixtures. The finished product meets all standard material specifications and construction requirements for the type of mixture being produced.
4. Recycling Agent—an organic material with special chemical and physical characteristics selected to restore aged asphalt to desired specifications.

Thickness Design

The thickness design procedures for a pavement structure employing recycled hot mix is no different than that required for a pavement using all new materials. This is applicable if the recycled mix meets mix design requirements for virgin mix. The thickness design methods contained in Chapter 11 are recommended. The procedure requires estimates of the anticipated traffic, environmental conditions, subgrade strength, and properties of the recycled mixture.

In many cases increased traffic weight and volume will require a stronger pavement section than was originally constructed. This can be done economically by designing a Full-Depth asphalt pavement using reclaimed untreated aggregate base materials in the recycled asphalt concrete mixture.

Any drainage deficiencies in the old pavement structure must be corrected before reconstruction proceeds. If any portion of the old subbase or base is to remain, the properties of these materials must be evaluated and appropriate layer coefficients assigned to them for use in the thickness design. Layer coefficients for effective thickness of existing layers can be found in Chapter 11.

12.2 Mix Design

Introduction

This section presents the step-by-step process necessary to proportion the reclaimed materials, select the grade and quantity of asphalt cement (plus recycling agent, if needed) and prepare a final design for the recycled mixture. This is the hot-mix method of recycling, using from 10 to 60 percent reclaimed asphalt pavement. Batch plants can handle up to 40 percent (without some auxiliary method of preheating RAP), with the most practical range being 10 to 35 percent; drum-mix plants can handle up to 60 percent, with 10 to 50 percent being a practical range.

Preparatory Steps

This mix-design procedure uses either the Marshall or the Hveem method, (see Chapter 4, also the Marshall and Hveem methods of mix design are fully described in *Mix-Design Methods for Asphalt Concrete and Other Hot-Mix Types,* MS-2, Asphalt Institute).

The aggregate from a reclaimed asphalt pavement (RAP) is blended with reclaimed aggregate materials and/or new aggregate that is required to obtain a combined aggregate gradation meeting the specification requirements. Once the relative aggregate proportions are determined, a total asphalt demand is calculated. A grade of new asphalt is then selected (plus recycling agent, if needed) to restore the aged asphalt and provide a final binder that meets the functional requirements of the asphalt specifications while satisfying the asphalt demand of the mix. Following these determinations, the mix design by either Marshall or Hveem procedure is performed and the exact quantity of total binder determined.

Materials Evaluation

The design of asphalt paving mixtures containing reclaimed asphalt pavement requires certain laboratory tests in addition to the usual Marshall or Hveem procedures. First, the composition of the reclaimed asphalt pavement must be determined. This will include:

(a) Aggregate gradation,
(b) Asphalt content,
(c) Asphalt viscosity at 60° C (140 ° F).

The aggregate and the asphalt in the reclaimed asphalt pavement have properties that must be evaluated separately. Therefore, it is necessary to extract the aged asphalt from a representative sample of the reclaimed asphalt pavement.

Then, the gradation of the reclaimed aggregate materials, if any, must be determined. This information is used to define the gradation and quantity of additional aggregates and the quantity of new asphalt cement that is needed.

(a) Aggregate Evaluation—A sieve analysis, ASTM C 117 and C 136 (AASHTO T 11 and T 27), is performed on the aggregate portion of the reclaimed asphalt pavement sample to determine the gradation. Any deficiencies can be corrected by blending appropriate sieve fractions of new and/or reclaimed aggregate with reclaimed asphalt pavement aggregate.

(b) Extraction—The method used should be ASTM Designation D 2172 (AASHTO T 164). The purpose of the extraction is quantitative separation of the aggregate and asphalt.

(c) Asphalt Evaluation—The asphalt content of the reclaimed asphalt pavement is determined on the basis of the relative weights of the extracted asphalt and aggregate. The extracted asphalt is reclaimed from solution by ASTM Method D 1856 (AASHTO T 170). Its consistency then is determined on the basis of viscosity at 60° C (140 °F), ASTM D 2171 (AASHTO T 202). This determination is necessary to estimate the required amount and grade of asphalt to be used in the recycled mix design.

It is suggested that when low percentages, not more than 10 percent, of reclaimed asphalt pavement (RAP) are to be incorporated in the mix, the viscosity of the extracted asphalt is not critical. The grade of asphalt to be used in the recycled mix will normally be the same grade used for conventional mixes.

New asphalt cement, added to the mixture of reclaimed asphalt pavement and new aggregate, serves two purposes. It increases the total asphalt content to meet the requirements of the mixture; and it blends with the aged asphalt in the reclaimed portion of the mixture to yield an asphalt meeting the desired specifications. Generally, AC-10, AC-5 or AC-2.5 (AR-4000, AR-2000 or AR-1000; 85-100, 120-150 or 200-300 pen.) asphalt cements are used for this purpose. These asphalts should meet standard specifications, ASTM D 3381 or D 946 (AASHTO M 226 or M 20).

Recycling agents are organic materials with chemical and physical characteristics selected to restore aged asphalt to desired specifications. In selecting the recycling agent, the viscosity characteristics of the combined aged asphalt and recycling agent are the determining factors. Specifications for recycling agents are contained in ASTM D 4552.

A number of recycling agents have been successfully used in the design of recycled mixtures.

Mix Design

With the information obtained from the materials evaluation, the recycled hot-mix design may be formulated. Viscosity at 60° C (140 ° F), ASTM Designation D 2171 (AASHTO T 202), is the test measurement used in this procedure to identify asphalt in the reclaimed asphalt pavement and in the recycled mixture.

The following steps can be used for the recycled mix design procedure:

1. Calculate combined aggregate in the recycled mix.
2. Approximate asphalt demand of combined aggregates.
3. Estimate the percent of new asphalt in the mix.
4. Select grade of new asphalt.
5. Trial mix designs by the Marshall or Hveem Method.
6. Select job mix formula.

An explanation of these steps is given in the following paragraphs.

1. Combined aggregates in the Recycled Mixture—Using the gradation of the aggregate from the reclaimed asphalt pavement, the reclaimed aggregate material, if any, and new aggregate, a combined gradation meeting the desired specification requirements is calculated.

After the blend of aggregate (aggregate in the RAP, new aggregates and/or RAM) have been established, the amount of new aggregate (and/or RAM) is expressed as r, in percent.

For example, suppose the following blend was established for a recycled mix:

45%	reclaimed aggregate (RAM)
20%	new aggregate
35%	RAP aggregate
100%	Total

The amount of new aggregate and RAM is 65%. Hence, $r = 65$. Table 12.1 contains formulas for proportioning materials for recycled asphalt hot mixes where the blend of aggregates in the mix is kept constant.

2. Approximate Asphalt Demand of the Combined Aggregates—The approximate asphalt demand of the combined aggregates may be determined by the Centrifuge Kerosene Equivalent (CKE) test included in the Asphalt Institute Hveem Method of Mix Design, or calculated by the following empirical formula:

$$P = 0.035a + 0.045b + Kc + F$$

where:

P = approximate total asphalt demand of recycled mix, percent by weight of mix
a = percent of mineral aggregate retained on 2.36 mm (No. 8) sieve
b = percent of mineral aggregate passing the 2.36 mm (No. 8) sieve and retained on the 75 μm (No. 200) sieve
c = percent of mineral aggregate passing 75 μm (No. 200) sieve
K = 0.15 for 11-15 percent passing 75 μm (No. 200) sieve
0.18 for 6-10 percent passing 75 μm (No. 200) sieve
0.20 for 5 percent or less passing 75 μm (No. 200) sieve
F = 0 to 2.0 percent. Based on absorption of light or heavy aggregate. In the absence of other data, a value of 0.7 is suggested

Table 12.1 Formulas for Proportioning Materials for Recycled Hot Mixtures
(Where blend of aggregates in the mix is kept constant)

		For Asphalt Content	
		by wt. of total mix	by wt. of aggregate
% New Asphalt,	P_{nb}	$\frac{(100^2 - r\ P_{sb})P_b}{100\ (100 - P_{sb})} - \frac{(100 - r)P_{sb}}{100 - P_{sb}}$	$P_b - \frac{(100 - r)P_{sb}}{100}$
% RAP,	P_{sm}	$\frac{100\ (100 - r)}{100 - P_{sb}} - \frac{(100 - r)P_b}{100 - P_{sb}}$	$\frac{(100 + P_{sb})(100 - r)}{100}$
% New Agg. and/or RAM,	P_{ns}	$r - \frac{rP_b}{100}$	r
Total		100	$100 + P_b$
% New Asphalt to Total Asphalt Content, R		$\frac{100\ P_{nb}}{P_b}$	$\frac{100\ P_{nb}}{P_b}$

P_{sm} = Percent salvage mix (RAP) in recycled mix
P_b = Asphalt content of recycled mix, %
P_{sb} = Asphalt content of salvaged mix (RAP), %
P_{nb} = Additional asphalt and/or recycling agent in recycled mix, %
P_{ns} = Percent additional aggregate (new or reclaimed aggregate material)
r = Percent new and/or reclaimed aggregate material to total aggregate in recycled mix
R = Percent new asphalt and/or recycling agent to total asphalt in recycled mix

With an approximate asphalt demand established, this will provide a basis for a series of trial mixes for a mix design. Trial mixes will usually vary in asphalt contents in 0.5 increments on either side of the calculated approximate asphalt demand.

For example, suppose that the approximate asphalt demand was calculated to be 6.2 percent. A series of trial mixes might then range from 5.0 to 7.0 percent or from 5.5 to 7.5 percent.

3. Estimated Percent of New Asphalt in Mix—The quantity of new asphalt to be added to the trial mixes of the recycled mixture, expressed as percent by weight of total mix, is calculated by the following formula:

$$P_{nb} = \frac{(100^2 - rP_{sb})P_b}{100(100-P_{sb})} - \frac{(100-r)P_{sb}}{100-P_{sb}}$$

where:

P_{nb} = percent of new asphalt* in recycled mix
r = new aggregate (and/or RAM) expressed as a percent of the total aggregate in the recycled mix
P_b = percent, asphalt content of reclaimed asphalt pavement asphalt demand, determined by CKE or empirical formula in item (2) above
P_{sb} = percent, asphalt content of reclaimed asphalt pavement

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**Plus recycling agent, if used.*

For example, suppose the asphalt content, P_{sb} of the RAP is 4.7 percent and r = 65%, then

$$P_{nb} = \frac{(100^2\text{-}65\times4.7)P_b}{100\ (100 - 4.7)} - \frac{(100\text{-}65)4.7}{100 - 4.7} = 1.01\ P_b\text{-}1.23$$

The percentages of new asphalt for any asphalt content may now be readily determined.

Note: The formula above is for asphalt content expressed as percent by weight of total mix. If asphalt contents are expressed as percent by weight of aggregate the formula for calculating quantity of new apshalt is:

$$P_{nb} = P_b - \frac{(100 - r)P_{sb}}{100}$$

(See Table 12.1)

(4) Select Grade of New Asphalt—Using Figure 12.1, a target viscosity of the asphalt blend is selected. A commonly selected target point is the viscosity at the mid-range or the normal paving grades, say an AC-20 asphalt or 2,000 poises.

The percent of the new asphalt, P_{nb}, to the total asphalt content, P_b, is expressed by the following formula:

$$R = \frac{100\ P_{nb}}{P_b}$$

For example, suppose the mix described in Step (3) is to have an estimated asphalt content of 6.2 percent. The amount of new asphalt to be added (from Step 3) is:

$P_{nb} = 1.01 \times 6.2 - 1.23 = 5.0$ percent

Then;

$$R = \frac{100\ (5.0)}{6.2} = 81$$

The grade of new asphalt (and/or recycling agent) is determined using a log-log viscosity versus percent new asphalt blending chart such as Figure 12.1. A target viscosity for the blend of recovered asphalt and the new asphalt (and/or recycling agent) is selected. The target velocity is usually the viscosity of the mid-range of the grade of asphalt normally used depending on type of construction, climatic conditions, amount and nature of traffic.

Plot the viscosity of the aged asphalt in the RAP on the left hand vertical scale, Point A, as illustrated in Figure 12.1. Draw a vertical line representing the percentage of new asphalt, R, calculated above and determine its intersection with the horizontal line representing the target viscosity, Point B. Then draw a straight line from Point A, through Point B and extend it to intersect the right hand scale, Point C. Point C is the viscosity at 60° C (140° F) of the new asphalt (and/or recycling agent) required to blend with the asphalt in the reclaimed asphalt pavement to obtain the target viscosity in the blend. Select the grade of new asphalt that has a viscosity range that includes or is closest to the viscosity at Point C.

When selecting the grade of the added asphalt, a grade not more than one grade lower than that normally used should be chosen. The exception is the case where engineering experience dictates that an even lower grade is acceptable.

5. Trial Mix Design—Trial mix designs are then made using the Marshall or Hveem apparatus. The formulas shown in Table 12.1 are used for proportioning the ingredients: new asphalt, P_{nb}, reclaimed asphalt pavement (RAP), P_{sm}, and new and/or reclaimed aggregate (RAM), P_{ns}.

Keep in mind that if two different aggregate sources are utilized, such as new aggregate and RAM, that the percentages of each of these sources must be determined and the total equal P_{ns}. For example the aggregate blend consists of

45% RAM
20% New aggregate
35% RAP aggregate
$r = 65$

If P_{ns} in a theoretical trial mix is to be 61.4 percent, then the percent of RAM (in the total mix) will be $61.4 \times (45/65) = 42.5$ percent and the percent new aggregate will be $61.4 \times (20/65) = 18.9$. The total equals 61.4 percent.

6. Select Job-Mix Formula - Refer to criteria for standard mixes in Chapter 4.

Design Example #1

The reclaimed asphalt pavement has an asphalt content of 5.4 percent by weight of total mix. The viscosity of the asphalt recovered from the reclaimed asphalt pavement is 46,000 poises at 60° C (140° F). The grade of asphalt cement normally used is AC-20, and the target viscosity at a temperature of 60° C (140° F) is 2,000 poises. Gradation of RAP, RAM and new aggregate is:

Sieve Size	Percent Passing RAP AGG.	 RAM	 NEW AGG.
25.0 mm (1 in.)	100	100	100
19.0 mm (¾ in.)	98	92	100
9.5 mm (⅜ in.)	85	45	100
4.75 mm (No. 4)	65	19	94
2.36 mm (No. 8)	52	5	85
300 μm (No. 50)	22	1	26
75 μm (No. 200)	8	0	6

Approximately 30 percent of RAP was selected because a batch plant was to be used for recycling, moisture content of the RAP was 5 percent and this is a practical range for maintaining mix productions.

Step 1—Combined Aggregates in Recycled Mixture

	30% RAP Aggr.	+	50% RAM Aggr.	+	20% NEW Aggr.	=	Comb. Aggr.
Sieve	% Pass.		% Pass.		% Pass.		% Pass.
25.0 mm (1 in.)	[100×0.3=30.0]	+	[100×0.5=50.0]	+	[100×0.2=20.0]	=	100.0
19.0 mm (¾ in.)	[98×0.3=29.4]	+	[92×0.5=46.0]	+	[100×0.2=20.0]	=	95.4
9.5 mm (⅜ in.)	[85×0.3=25.5]	+	[45×0.5=22.5]	+	[100×0.2=20.0]	=	68.0
4.75 mm (No. 4)	[65×0.3=19.5]	+	[19×0.5= 9.5]	+	[94×0.2=18.8]	=	47.8
2.36 mm (No. 8)	[52×0.3=15.6]	+	[5×0.5= 2.5]	+	[85×0.2=17.0]	=	35.1
300 μm (No. 50)	[22×0.3= 6.6]	+	[1×0.5= 0.5]	+	[26×0.2= 5.2]	=	12.3
75 μm (No. 200)	[8×0.3= 2.4]	+	[0×0.5= 0]	+	[6×0.2= 1.2]	=	3.6

Then: r = 50 + 20 = 70

Job Specification ASTM D 3515, (Table-1) ¾ in. (19mm) Nominal Size

Sieve Size	Max. Size % Pass.	Combined Aggr. % Pass.
25.0 mm (1 in.)	100	100.0
19.0 mm (¾ in.)	90 - 100	95.4
9.5 mm (⅜ in.)	56 - 80	68.0
4.75 mm (No. 4)	35 - 65	47.8
2.36 mm (No. 8)	23 - 49	35.1
300 μm (No. 50)	5 - 19	12.3
75 μm (No. 200)	2 - 8	3.6

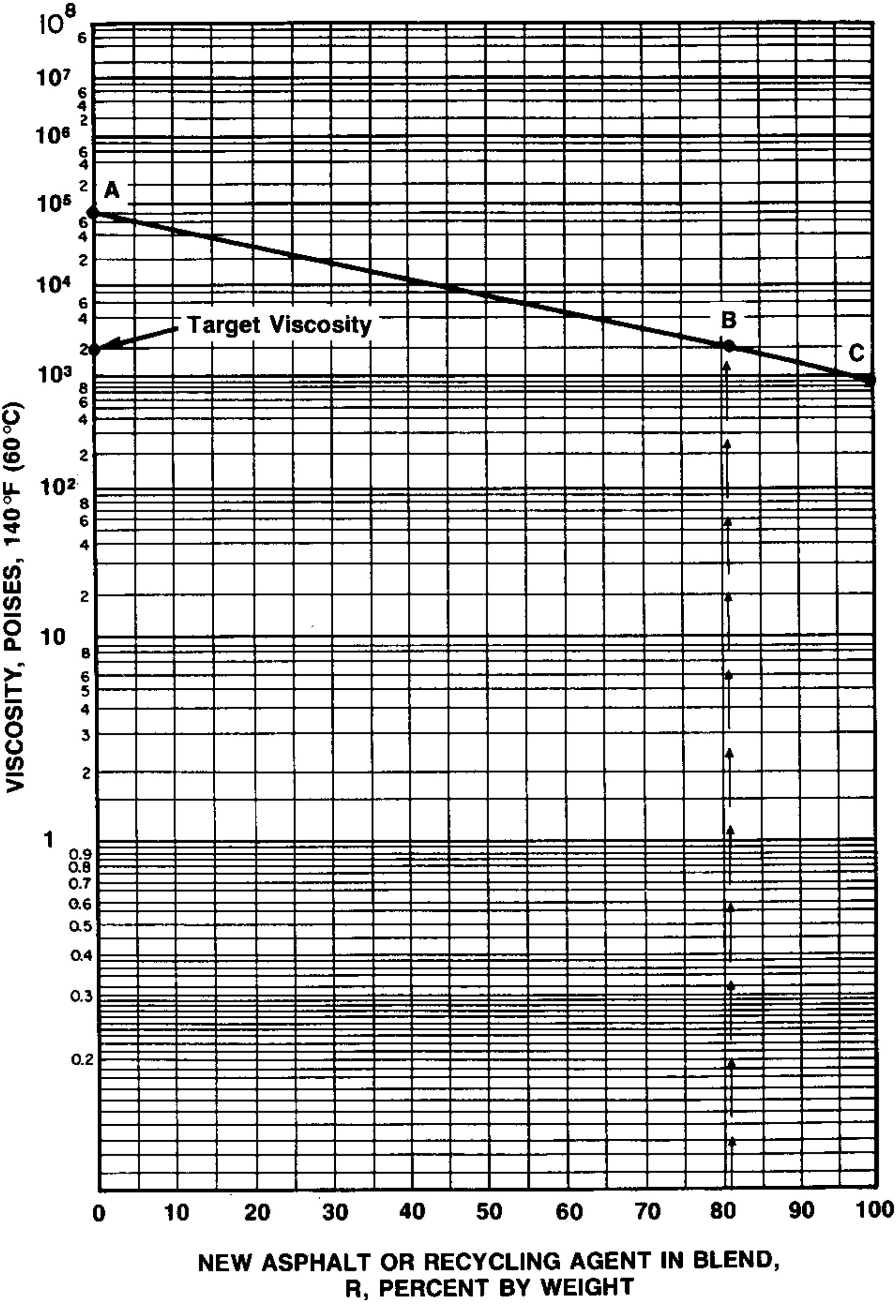


Figure 12.1 Asphalt Viscosity Blending Chart.

Step 2 - Approximate Asphalt Demand of Combined Aggregates

$$\begin{aligned} P &= 0.035a + 0.045b + Kc + F \\ &= 0.035 \times 64.9 + 0.045 \times 31.5 + 0.20 \times 3.6 + 0.4 \\ &= 5.2 \text{ percent} \end{aligned}$$

Step 3 - Estimated Percent of New Asphalt in Mix

$$P_{nb} = \frac{(100^2 - r\,P_{sb})P_b}{100\,(100 - P_{sb})} - \frac{(100 - r)P_{sb}}{100 - P_{sb}}$$

$$= \frac{(100^2 - 5.4 \times 70)P_b}{100\,(100 - 5.4)} - \frac{(100 - 70)5.4}{100 - 5.4}$$

$$= 1.02\,P_b - 1.71$$

For an approximate asphalt demand of 5.2 percent (Step 2);

$$P_{nb} = 1.02\,(5.2) - 1.71 = 3.6 \text{ percent}$$

The percent of new asphalt, P_{nb}, to total asphalt, P_b, will then be;

$$R = \frac{100\,(3.6)}{5.2} = 69 \text{ percent.}$$

Step 4 - Select Grade of New Asphalt

On Figure 12.2 Point A is the viscosity of the aged asphalt at 46,000 poises (4.6×10^4). Point B is located from a target viscosity of 2,000 poises (2.0×10^3) and R=69. The projected line from Point A through Point B to Point C indicated that the viscosity of the new asphalt is 7.0×10^2 poises (700).

Since AC-20 is the normal grade of asphalt cement used in the area of construction, climate and traffic, an AC-10 will be chosen for this project. The AC-10 when blended with the aged asphalt in the RAP should result in an AC-20 within acceptable tolerances.

Step 5 - Trial Mix Design

Using an aggregate blend of 50 percent RAM, 20 percent new aggregate and 30 percent RAP aggregate, trial mixes of different asphalt contents (varying in 0.5 percent increments) are prepared according to standard Marshall or Hveem mix design procedures.

The formulas in Table 12.1 may be used to determine the percentages of each ingredient in the trial mixes. Since the formula for P_{nb} was calculated in Step 3, the formulas for proportioning the P_{sm} and P_{ns} are:

$$P_{sm} = \frac{100\,(100 - r)}{100 - P_{sb}} - \frac{(100 - r)P_b}{100 - P_{sb}}$$

$$= \frac{100\,(100 - 70)}{100 - 5.4} - \frac{(100 - 70)P_b}{100 - 5.4}$$

$$= 31.71 - 0.32\,P_b$$

$$P_{ns} = r - \frac{r\,P_b}{100}$$

$$= 70 - \frac{70P_b}{100} = 70 - 0.70P_b$$

Asphalt Content, P_b	4.5	5.0	5.5	6.0	6.5
$P_{nb} = 1.02\,P_b - 1.71$	2.9	3.4	3.9	4.4	4.9
$P_{sm} = 31.71 - 0.32\,P_b$	30.3	30.1	29.9	29.8	29.6
$P_{ns} = 70 - 0.70\,P_b$	66.8	66.5	66.2	65.8	65.5
Total	100.0	100.0	100.0	100.0	100.0
*% RAM = P_{ns} (50/70)	47.8	47.5	47.3	47.0	46.8
*% New Aggr = P_{ns} (20/70)	19.1	19.0	18.9	18.8	18.7

**The percentages of new aggregate and RAM as a blend were determined as P_{ns}. However, 50 percent RAM and 20 percent new aggregate are to be used in the aggregate blend. The amount of RAM will then be P_n × 50/70 and the new aggregate will be P_{ns} × 20/70.*

When preparing trial mixes in the laboratory, it is suggested that the RAP be heated to mixing temperature and maintained at that temperature. The new aggregate and RAM are normally heated to 20° C (50° F) above the mixing temperature. When the aggregate and RAP have been weighed out, dry mixing should begin to thoroughly blend the materials before adding new asphalt. Keeping the RAP at elevated temperatures should be held to a minimum. Otherwise, normal mix design procedures are followed.

Step 6 - Select Job-Mix Formula

The optimum new asphalt content and the mix design are determined according to established standard Marshall or Hveem mix-design criteria (as is used for virgin materials).

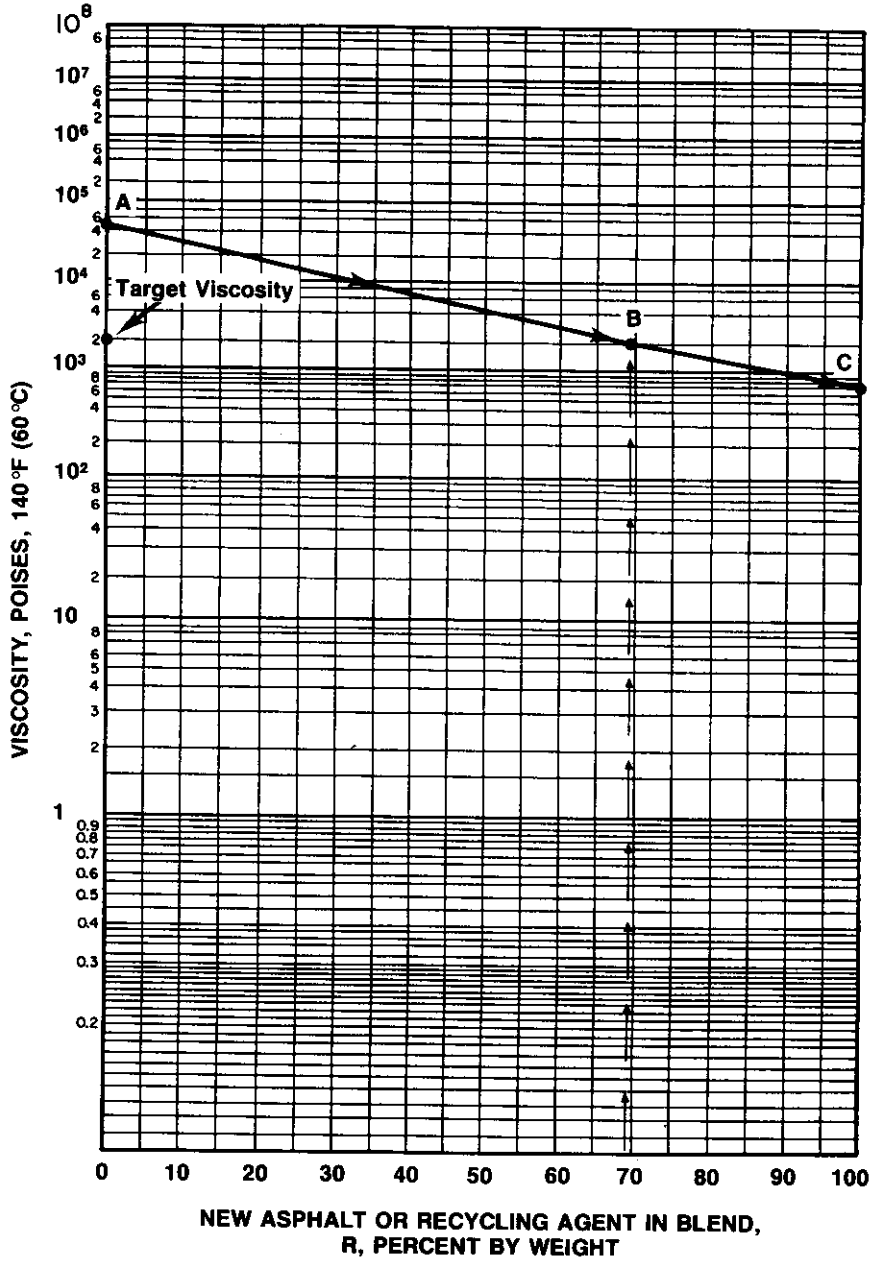


Figure 12.2 Asphalt Viscosity Blending Chart (Design Example 1).

Design Example 2

Reclaimed asphalt pavement has an asphalt content of 6.0 percent with a viscosity of 100,000 poises. Gradation or RAP, RAM and new aggregate are the same as for Example 1.

Steps 1 and 2 - Same as Example 1.

Step 3 - Estimated Percent of New Asphalt in Mix

$$P_{nb} = \frac{(100^2 - r\ P_{sb})P_b}{100\ (100 - P_{sb})} - \frac{(100 - r)P_{sb}}{100 - P_{sb}}$$

$$= \frac{(100^2 - 70\times 6.0)P_b}{100\ (100 - 6.0)} - \frac{(100 - 70)6.0}{100 - 6.0}$$

$$= 1.02\ P_b - 1.91$$

For an approximate asphalt demand of 5.2 (Step 2) percent;

$$P_{nb} = 1.02\ (5.2) - 1.91 = 3.4 \text{ percent}$$

Step 4 - Select Grade of New Asphalt

On Figure 12.3 Point A is the viscosity of the aged asphalt at 100,000 poises (1.0×10^5). Point B is located using values of 2,000 poises (2.0×10^3) for target viscosity and R=57, ($100P_{nb}/P_b = 100\times3.4/6.0$) of new asphalt. A line is projected through these two points and intersects the right axis at 1.8×10^2 (180 poises), Point C.

This is a heavily-traveled roadway where the design engineer is concerned with channeling and normally uses an AC-20 in mix design. Figure 12.3 can be used to determine how much of a recycling agent to blend with AC-20 to give an apparent viscosity of 180 poises.

Let the AC-20 be the new asphalt and plot 2,000 poises (2.0×10^3) on the left-hand scale, Point D. The viscosity of the recycling agent is 1 poise. Plot this as Point E on the right-hand scale. Connect Points D and E with a straight line. Now determine what percentage, R, of recycling agent will be required to result in a viscosity of 180 poises for the blend. This is plotted as Point F on the line from D to E. The percentage R on the horizontal scale indicates 22 percent. This means that a tank of AC-20 containing 22 percent of the recycling agent should have a viscosity of approximately 180 poises. When this blend is added to the mix for a total asphalt content of about 5.2 percent, the viscosity of the total asphalt in the recycled mix should be 2,000 poises—within acceptable limits.

Step 5 - Trial Mix Design

Using an aggregate blend of 50 percent RAM, 20 percent new aggregate and 30 percent RAP aggregate, trial mixes of different asphalt contents (varying in 0.5 percent increments on either side of the estimated asphalt demand) are prepared according to standard Marshall or Hveem mix design procedures.

The formulas in Table 12.1 may be used to calculate the percentages of each ingredient in the trial mixes. Since the formula for P_{nb} was calculated in Step 3, the formulas for proportioning P_{sm} and P_{ns} are:

$$P_{sm} = \frac{100\,(100 - r)}{100 - P_{sb}} - \frac{(100 - r)P_b}{100 - P_{sb}}$$

$$= \frac{100\,(100 - 70)}{100 - 6} - \frac{(100 - 70)P_b}{100 - 6}$$

$$= 31.91 - 0.32\,P_b$$

$$P_{ns} = r - \frac{r\,P_b}{100}$$

$$= 70 - \frac{70P_b}{100} = 70 - 0.70P_b$$

Asphalt Content, P_b	4.0	4.5	5.0	5.5	6.0
$P_{nb} = 1.02\,P_b - 1.91$	2.2	2.7	3.2	3.7	4.2
$P_{sm} = 31.91 - 0.32\,P_b$	30.6	30.5	30.5	30.1	30.0
$P_{ns} = 70 - 0.70\,P_b$	67.2	66.8	66.5	66.2	65.8
Total	100.0	100.0	100.0	100.0	100.0
*% RAM = P_{ns} (50/70)	48.0	47.7	47.5	47.3	47.0
*% New Aggr. = P_{ns} (20/70)	19.2	19.0	19.0	18.9	18.8

**The percentages of new aggregate and RAM as a blend were determined as P_{ns}. However, 50 percent RAM and 20 percent new aggregate are to be used in the aggregate blend. The amount of RAM will then be P_{ns}×50/70 and the new aggregate will be P_{ns}×20/70.*

When preparing trial mixes in the laboratory, it is suggested that the RAP be heated to and maintained at the mixing temperature. The aggregate is normally heated to mixing temperature plus 20° C (50° F). When the aggregate and RAP have been weighed out, dry mixing should begin to thoroughly blend the materials before adding new asphalt. Keeping the RAP at elevated temperatures should be held to a minimum. Otherwise, normal mix design procedures are followed.

Step 6 - Select Job-Mix Formula

The optimum new asphalt content and the mix design are determined according to established standard Marshall or Hveem mix-design criteria (as is used for virgin materials).

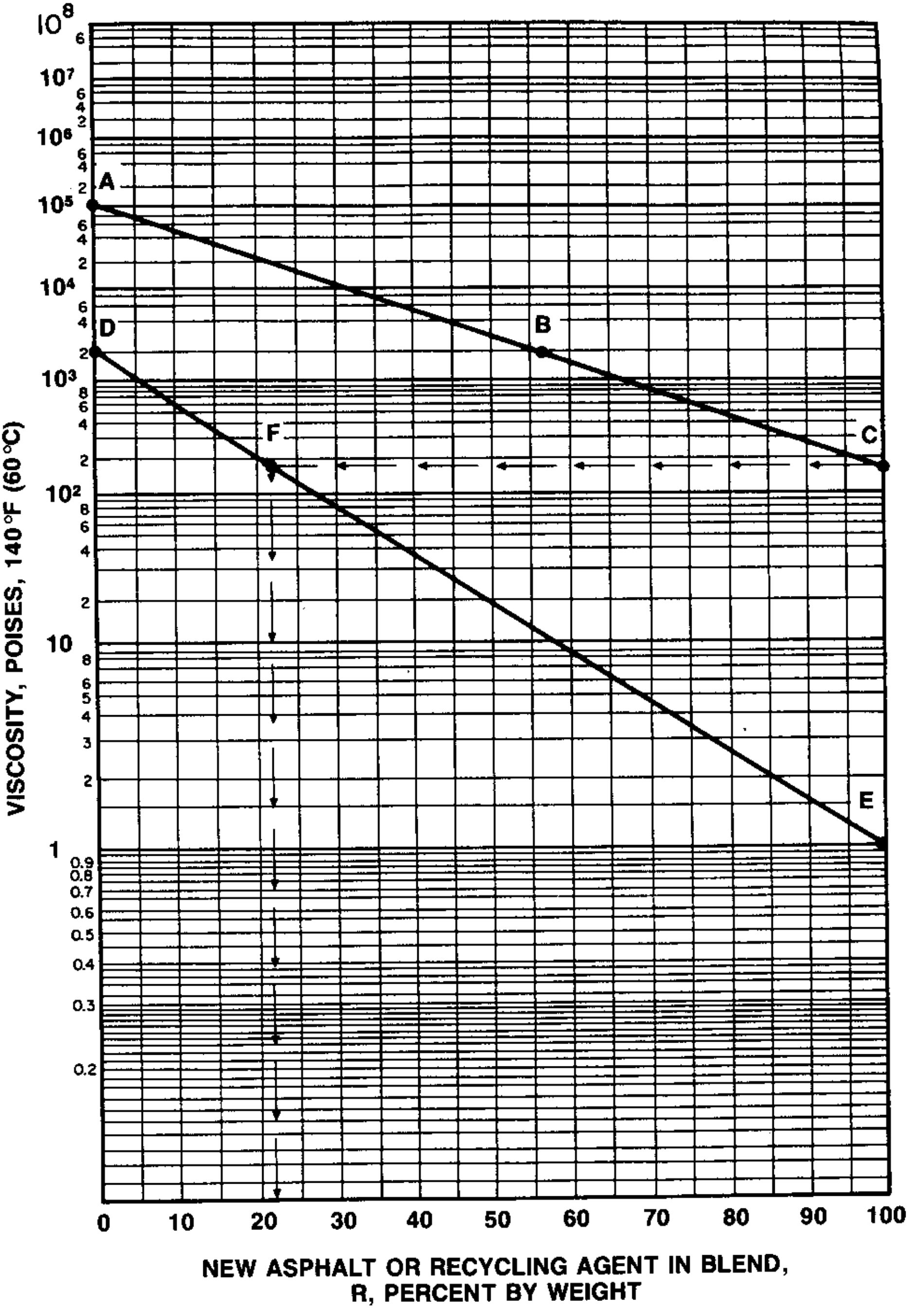


Figure 12.3 Asphalt Viscosity Blending Chart (Design Example 2).

12.3 Reclaimed Materials

Pavement Removal

The most commonly used methods for pavement removal are ripping and crushing, and cold milling or planing of the old pavement. Regardless of the method used, it is essential that the reclaimed asphalt pavement be kept separated from the reclaimed untreated aggregate material. Any intermingled materials that do result from the removal process should not be stockpiled with or added to the reclaimed asphalt pavement.

(a) *Reclaimed Asphalt Pavement*—In the ripping and crushing operation, scarifiers, grid rollers, or rippers are used to break up the asphalt pavement. Then it is loaded and hauled to a crushing and screening plant (Figure 12.4). Alternatively, the broken-up pavement is pulverized on the roadway by a hammermill or by additional passes of grid or V-cleated roller. Proper material sizing and separation of reclaimed asphalt pavement from the reclaimed aggregate material is more difficult in the latter case.

Specialized cold milling or planing machines can reclaim asphalt pavement to controlled depths. In the process, the pavement is reduced to the desired maximum particle size. Particle size is dependent upon the depth of the cut and forward speed of the cold milling equipment as well as specifics of the cutting teeth, (See Figures 12.5 and 12.6).

(b) *Reclaimed Aggregate Material*—After the asphalt-treated layers have been removed, any remaining aggregate materials that are to be incorporated in the recycled hot-mix may be scarified and removed with loaders, grade trimmers, or other conventional equipment. This material must be kept in a separate stockpile apart from the reclaimed asphalt pavement material.

When the pavement material removal is completed, any drainage deficiencies should be corrected. After that, the subbase or subgrade is cut, graded, and compacted to proper grade, cross section, and profile.

Pavement Size Reduction

The degree to which reclaimed pavement materials must be processed after removal depends largely on the removal method, the characteristics of the mixing equipment, and the requirements of the mix design. Reclaimed asphalt pavement and reclaimed aggregate materials must be processed separately. Asphalt pavement removed by ripping will have to be crushed and screened. Some reclaimed pavement materials may require crushing to reduce the maximum particle size to acceptable limits, typically less than 50 mm (2 in.). Special machines, such as the one shown in Figure 12.4, have been designed for the specific purpose of crushing reclaimed asphalt pavement. None of the fines should be removed from reclaimed asphalt pavement since they contain much of the aged asphalt that is to be recycled. Also, it is important to note that the crusher used should minimize production of fine size material or new fractured faces. Changes could result in increased mix cost, potential processing problems or even selecting a different aggregate to provide suitable VMA.

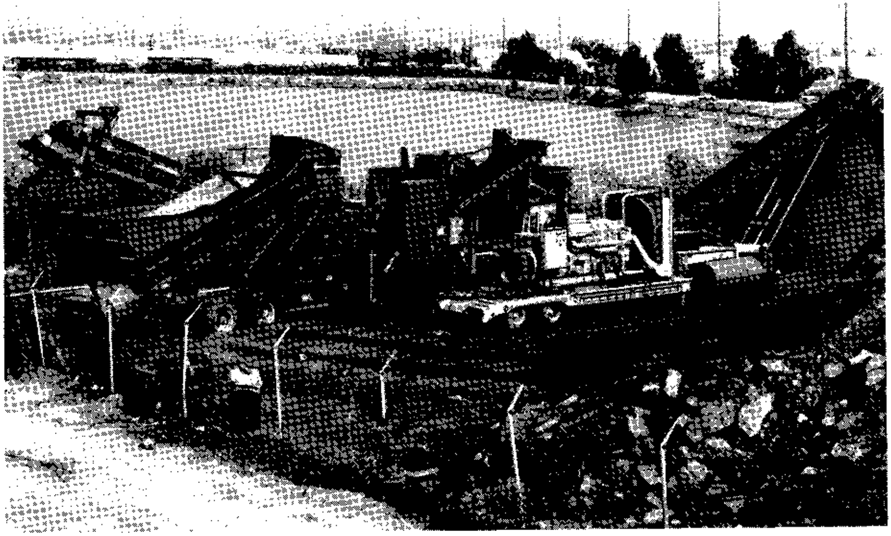


Figure 12.4 Reclaimed Asphalt Pavement Crusher *(Courtesy of Barber-Greene Company).*

Stockpiling

The height of reclaimed asphalt pavement stockpiles should be limited to 3 metres (10 feet) maximum. The restriction is to prevent the crushed materials from sticking together because of dead load and high air temperatures. For the same reason, loaders, dozers, and trucks should not be permitted on the stockpile. The reclaimed asphalt pavement should be protected from the weather to keep it as dry as possible. The best way to minimize both stocking and excessive moisture is to coordinate the crushing and hot-mixing operations so that a large stockpile of crushed asphalt pavements is not necessary. If, however, the reclaimed asphalt pavement tends to stick together or reconsolidate after a period of prolonged storage, the welding of steel spikes onto the front of a loader bucket will help to break up the RAP as it is loaded into the bucket.

Also, the finer character of the processed RAP material has a tendency to hold moisture and therefore requires greater consideration in protection from rain and snow. Dependent upon the annual moisture in the region, a

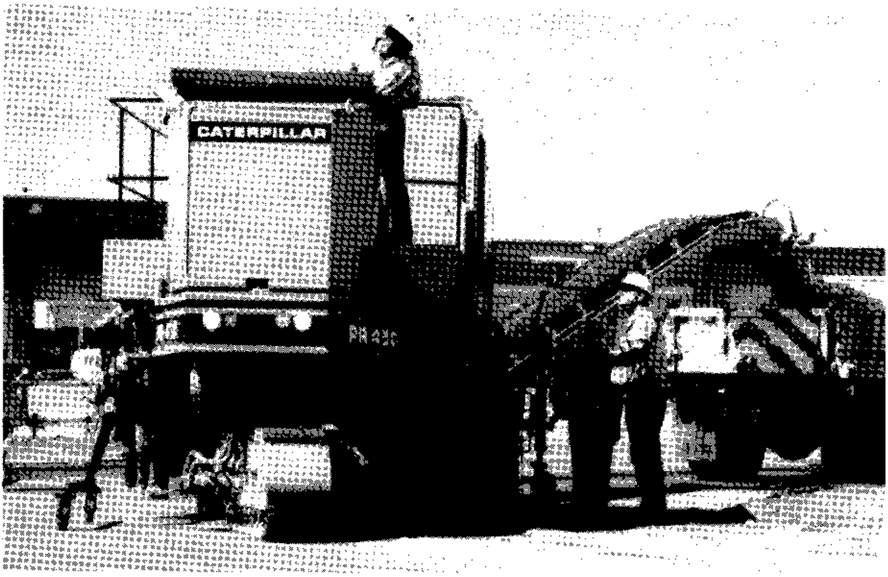


Figure 12.5 Cold-Milling *(Courtesy of Caterpillar Tractor Co.).*

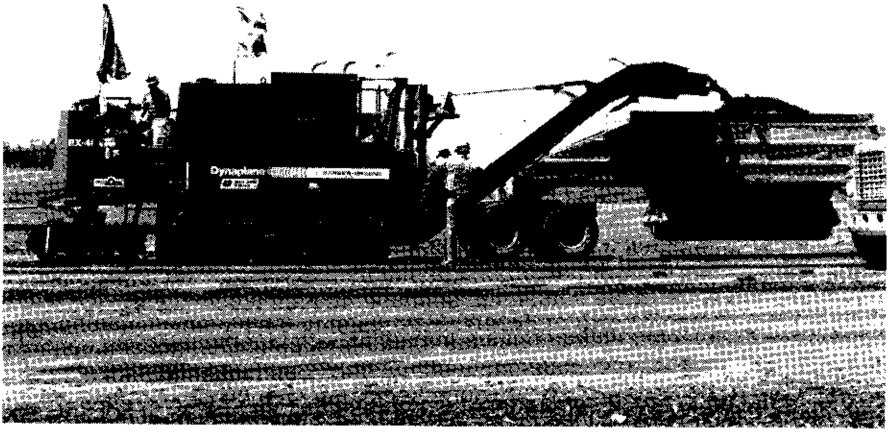


Figure 12.6 Cold-Milling *(Courtesy of Barber-Greene Company).*

protective covering may be cost-effective in protecting the crushed RAP material whether the source is cold milled or crushed. A cost analysis will determine the most effective protective procedure.

Reclaimed untreated aggregate base and subbase should be stockpiled in the same manner as new aggregate.

12.4 Facility Production Methods

Production Options

When reclaimed asphalt pavement is part of the recycled mixture, some changes in normal mixing facility processes are necessary. A number of procedures have been developed. The objective of each is to heat and dry the reclaimed asphalt pavement without exposing it directly to the high temperature flame and combustion gases in the dryer. Without these changes, recycled mixtures cannot be produced economically and still comply with regulations governing exhaust stack emissions. Equally important is protection from the asphalt in the reclaimed asphalt pavement from further hardening.

Both batch and drum-mix facilities have been modified for the successful production of hot recycled mixtures. Also, equipment manufacturers produce plants designed expressly for recycling. Because of the variety of procedures involved, each of the asphalt hot-mix facilities will be discussed separately.

Batch Facilities

The only technique that has proven successful in recycling through a batch plant is the heat-transfer method. Briefly, here is how it works.

The reclaimed aggregate material, or new aggregate, or both are proportioned from the cold feed bins. They are then heated in a conventional aggregate dryer. From there they are conveyed to the hot storage bins in the usual manner. (For asphalt hot-mix facilities equipped with baghouse dust collectors, extremely high exhaust temperatures could damage the bags. At exhaust gas temperatures over 232° C (450° F), the danger of bag deterioration is extreme.)

Without heating or drying, the reclaimed asphalt pavement is carried directly from the stockpile to the cold feed bin. It is necessary to have a separate cold feed bin that has steep sides (approaching vertical). The reclaimed material is then conveyed to the weigh hopper (See Figure 12.7). (It is weighed as a fifth material in a normal four-bin batch plant.) There it joins the superheated untreated aggregate. Heat-transfer takes place as the proportioned materials are mixed in the pugmill.

Complete temperature equilibrium in the hot recycled mixture usually is attained some time after the mix leaves the pugmill; storage silos are an aid in this respect.

This batch-plant heat-transfer method minimizes the chance of air pollution; it eliminates the problem of clogged screens; and it prevents reclaimed asphalt pavement build-up on the hot-stone elevator. However, sometimes a steam cloud is generated as moisture is driven from the

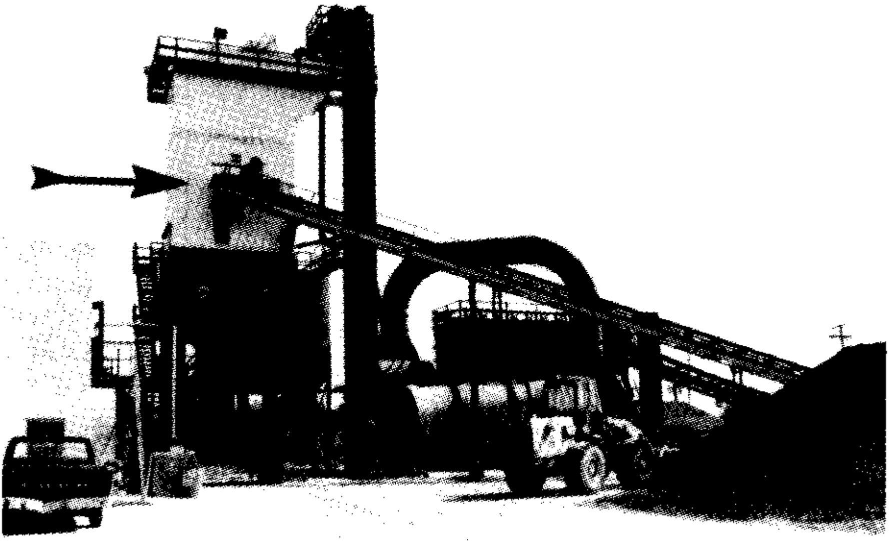


Figure 12.7 Recycling in Batch Facility; Arrow Indicates Entry Into Weigh Hopper *(Courtesy of Barber-Greene Company).*

reclaimed asphalt pavement material; when this occurs, venting of the pugmill may be advisable.

The amount of reclaimed asphalt pavement that can be used in the recycled mix depends on (1) the moisture content and stockpile temperature of the reclaimed material, (2) the required temperature of the recycled mix, and (3) the temperature of the superheated aggregate. If the conditions are right, i.e., moisture content of the reclaimed asphalt pavement is at, or near, a minimum and its temperature is close to the surrounding air temperature, as much as 40 percent of the mixture can be reclaimed asphalt pavement. However, a more realistic value is 20 to 30% for normal operations.

Drum-Mix Facilities

In the normal operation of a drum-mix plant, aggregates are heated, dried, and mixed with asphalt in the mixing drum. Early attempts at hot-mix recycling with these facilities usually produced satisfactory mixes but the plants were unable to meet minimum air-pollution standards. Exposure of the reclaimed asphalt pavement to the burner flame and the extremely hot combustion gases caused excessive blue smoke. Frequently there was a build-up of aggregate fines and asphalt on metal flights and end plates that also contributed to the smoke problem.

Many improvements have recently been made in drum mix recycling plants to accommodate variations in materials as well as to improve thermal efficiency. Several manufacturers produce drum mixers using the split feed approach. This is the technique most commonly used in the United States. In these mixing facilities, untreated aggregate enters the drum at the burner end of the drum. The aggregate is dried and superheated.

The aggregate joins the reclaimed asphalt pavement at a point far enough down-stream from the burner to be shielded from the flame and extremely hot gases. The reclaimed asphalt pavement enters the drum near the center through openings in the shell with chutes that channel the flow of material into the drum. New asphalt cement, recycling agent, or both and mineral filler, if needed are added. Mixing is accomplished in the lower half of the drum. (Figures 12.8, 12.9 and 12.10.)

At least one manufacturer has developed a facility that does not expose the asphalt to steam in the drum. Called a drum mix coater, this plant uses a single-shaft coater-mixer at the drum discharge. Untreated aggregate and reclaimed asphalt pavement are premixed in the drum and discharged by gravity into the coater, where the new asphalt is injected. (Figure 12.11).

Counter-flow drum mixers are also well suited to recycling. Details and illustrations of this operation are presented in Chapter 5.

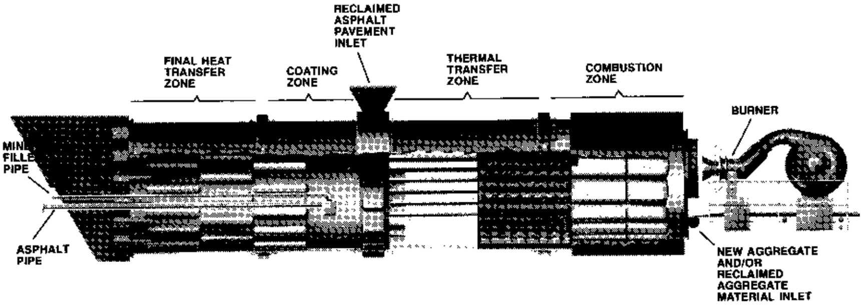
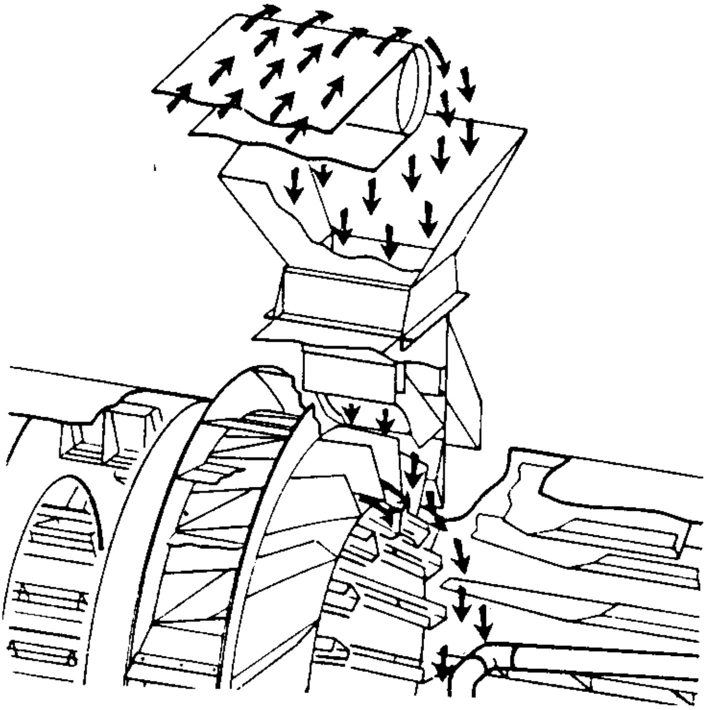


Figure 12.8 Drum Mixer *(Courtesy of Barber-Greene Company).*



RAP FLOW

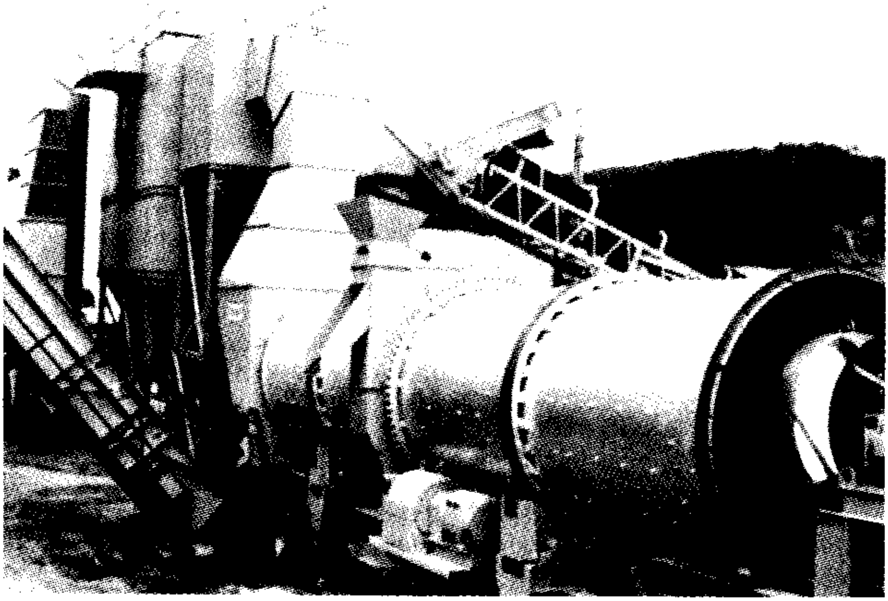


Figure 12.9 Drum Mixer *(Cedarapids—Courtesy of Iowa Manufacturing Co.).*

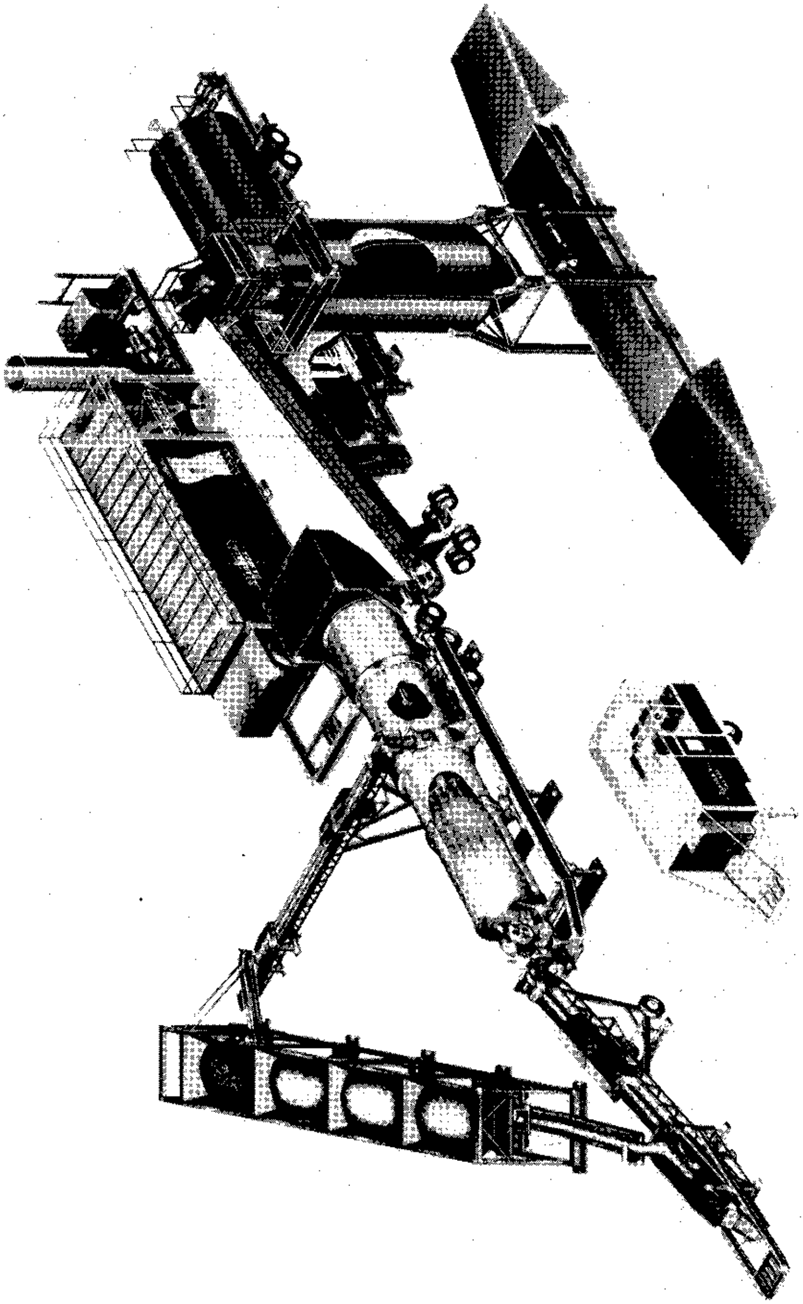


Figure 12.10 **Drum Mixer Facility Layout** *(Courtesy of Standard Havens).*

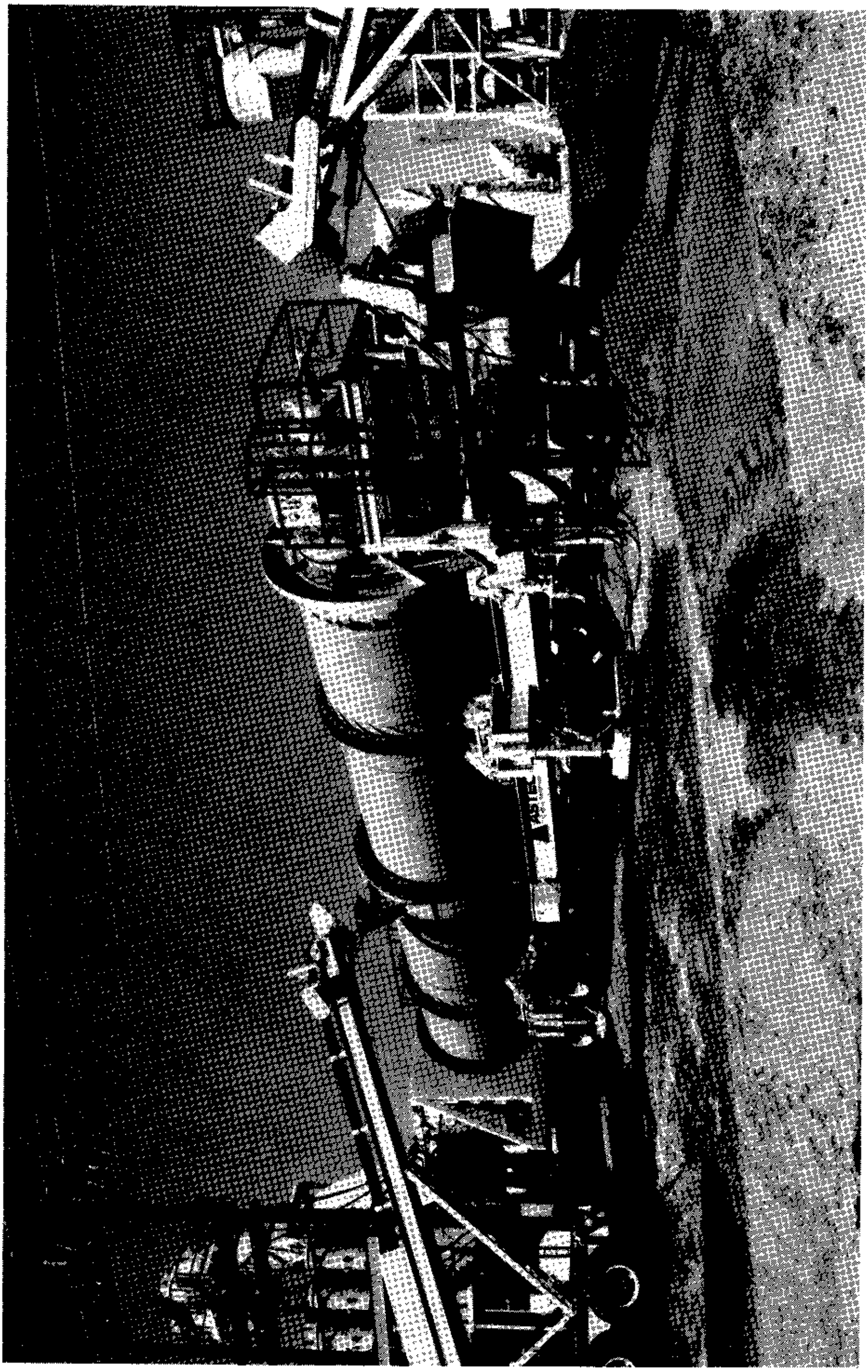


Figure 12.11 **Drum Mixer** *(Courtesy of ASTEC Industries, Inc.).*

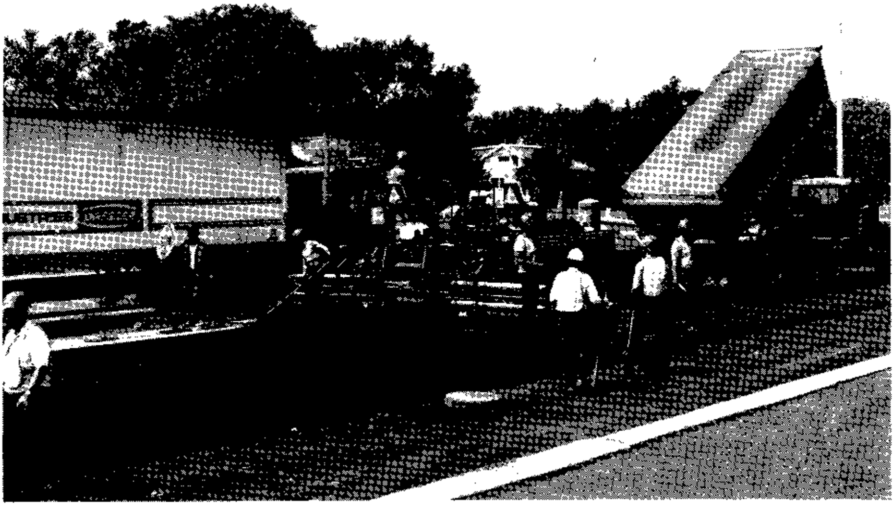


Figure 12.12 Spreading Reclaimed Hot-Mix Asphalt.
(Courtesy of Barber-Greene Company).

Spreading and Compacting

Conventional equipment and procedures are used for spreading and compacting hot-mix recycled mixtures (Figure 12.12). Spreading temperatures generally range between 104° and 138° C (220° and 300° F). Further details of these operations are contained in Chapters 6 and 7.

Chapter 13
Cold-Mix Asphalt

Cold-mix asphalt preparation is not as sophisticated as that described for hot mixes in Chapter 5. Generally, placing and compacting do not normally call for equipment as sophisticated as that required for hot-mix asphalt. This type of pavement construction, however, is no less important since a large percentage of the nation's roadways have cold-mix asphalt bases and even surfaces.

Less control is required for cold-mix preparation with a distributor and motor-grader operation. But possibilities for as great a control as for hot-mix asphalt are present in a plant-mix operation. However, many plants used for producing cold mixes do not use or have all the means for maintaining close uniformity of control. Wherever suitable materials are available, such plant mixing procedures are employed for high production and low cost.

Cold-mix recycling or reworking of existing asphalt pavement or aggregate structures is another important pavement construction alternative. Cold-mix recycling is the process in which reclaimed asphalt material, reclaimed aggregate materials or both are combined with new asphalt in place or at a central mixing facility to produce cold-mix base mixtures.

SECTION 13.1 Cold-Mix Asphalt Pavements
☐ Introduction
☐ Mix Composition
☐ Types of Mixes
☐ Construction Methods

SECTION 13.2 Cold-Mix Recycling
☐ In-Place Cold Recycling
☐ Central Facility Cold Recycling

BIBLIOGRAPHY

1. *Asphalt Cold-Mix Manual, MS-14,* Asphalt Institute.
2. *Asphalt Cold-Mix Recycling Manual, MS-21,* Asphalt Institute.

13.1 Cold-Mix Asphalt Pavements

Introduction

Description

Cold placed asphalt pavement mixtures (or simply cold mixes) are generally mixes made with emulsified or cutback asphalt. Emulsified asphalts may be anionic or cationic MS and SS grades. Cutback asphalts may be MC or SC grades.

Aggregate material may be anything from a dense-graded crushed aggregate to a granular soil having a relatively high percentage of dust. The aggregate at the time of mixing may be either damp, air-dry, or artificially heated and dried.

Mixing methods may be performed either in the roadway, along the roadway, or in a stationary mixing facility. The resulting mixtures usually are spread and compacted at atmospheric temperature, however, there are exceptions. Some asphalt mixes made with MC- and SC-3000 are similar to hot asphalt mixes because they are mixed and placed at temperatures in excess of 93° C (200° F). Also, some emulsion mixtures might be produced either warm or hot.

Suitability for Pavement Construction

Cold-laid asphalt mixes may be used for surface, base, or subbase courses if the pavement structure is properly designed. Cold-laid surface courses are suitable for light and medium traffic. When used in the base or subbase, they may be suitable for all types of traffic. Cold-laid pavements have special advantages in outlying areas where economic problems exist in getting the necessary improved mileage. Also used on large jobs, cold-laid paving mixtures reduce construction costs, but not necessarily the strength or quality of the pavement structure.

Asphalt pavements made by cold-laid methods can be designed and built to meet current traffic requirements. They can also be increased in thickness, as may be dictated for subsequent traffic, by adding additional pavement courses.

Mix Composition

A wide variety of aggregates and soil-aggregate combinations can be processed. There are many cases where material from road cuts, pits, or nearby quarry sites may be economically crushed or used. Often the entire output below a certain maximum size from a rock crusher is used.

Suitable local materials also include granular soils having up to 20 percent passing the 75 μm (No. 200) sieve, as well as clean sands and gravels.

Granular materials that are mostly fine aggregate and have relatively high percentage mineral dust may cause mixing, aeration, and compaction problems.

Well-graded aggregates are always desirable for any course of the asphalt pavement structure, but many poorly graded and gap-graded aggregates are adequate for base courses when properly prepared by road mixing methods and procedures.

Asphalt

The type and grade of asphalt selected for use in cold-laid pavement construction depends a great deal upon the aggregate, mixing method, and climatic conditions. Recommended asphalts for different types of cold-laid pavement mixtures are given in Table 13.1.

Mixtures prepared at a stationary mixing facility generally permit a more viscous grade of asphalt than used in the road-mixing method. Cool weather conditions require a lower viscosity grade and permit a faster curing or setting type of asphalt to be selected than for warmer or hot weather mixing conditions.

Mixtures prepared with emulsified asphalt do not require aggregates as dry as those prepared with cutback asphalts. This reduces the amount of manipulation or extent of air-drying prior to mixing the aggregate with asphalt.

Cold-laid asphalt mixtures prepared for maintenance use and stockpiled over a period of time usually are prepared by using one of the lower viscosity grades of the MC or SC cutback asphalts.

Proportioning the Materials

The amount of asphalt for cold-laid paving mixtures may be determined by one of the following laboratory procedures:

1. Methods of Test for Resistance to Deformation and Cohesion of Bituminous Mixtures by Means of Hveem Apparatus, ASTM D 1560; and
2. Centrifuge Kerosene Equivalent (CKE) method described in Chapter 4.

Where laboratory equipment is not available, formulas may be used for approximating the asphalt content for initial mixing.

For estimating the approximate emulsified asphalt content in dense-graded mixes the following empirical formula may be used:

$$P = (0.05A + 0.1B + 0.5C) \times (0.7) \qquad (1)$$

where:

P = percent of total emulsified asphalt by weight of dry aggregate,
A = percent of aggregate retained on the 2.36 mm (No. 8) sieve,
B = percent of aggregate passing 2.36 mm (No. 8) and retained on 75 μm (No. 200) and
C = percent of aggregate passing 75 μm (No. 200).

(All above percentages should be expressed as whole numbers.)

Table 13.1 Guide for Uses of Asphalt in Cold Mix

Type of Construction	Emulsified Asphalts													Cutback Asphalts						
	Anionic									Cationic				Medium Curing (MC)				Slow Curing (SC)		
	MS-1	MS-2	MS-2h	HFMS-1	HFMS-2	HFMS-2h	HFMS-2s	SS-1	SS-1h	CMS-2	CMS-2h	CSS-1	CSS-1h	70	250	800	3000	250	800	3000
Cold Plant Mix																				
Pavement Base & Surface																				
Open-Graded Aggregate	×	×	×	×	×	×				×	×					×				
Well-Graded Aggregate							×	×	×			×	×		×	×	×	×	×	×
Patching, Immediate Use								×	×			×	×		×	×			×	
Patching, Stockpile															×	×		×	×	
Mixed-in-Place (Road Mix)																				
Pavement Base & Surfaces																				
Open-Graded Aggregate	×	×	×	×	×	×				×	×					×	×		×	×
Well-Graded Aggregate							×	×	×			×	×		×	×		×	×	
Sand							×	×	×			×	×	×	×	×				
Sandy Soil							×	×	×			×	×		×	×				
Patching, Immediate Use							×	×	×			×	×		×	×			×	
Patching, Stockpile															×	×		×	×	

Example:

Given the following aggregates and construction method, determine the required amount of emulsified asphalt:

Method of construction: blade mixing;

Aggregate: relatively non-absorptive gravel, gradation shown below, and;

Asphalt: SS-1

Sieve size	19.0 mm (¾″)	9.5 mm (⅜″)	4.75 mm (No. 4)	2.36 mm (No. 8)	600 m (No. 30)	300 μm (No. 50)	150 μm (No. 100)	75 μm (No. 200)
Percent passing	100	75	59	40	20	14	10	5

Solution:

$$P = [0.05\,(100 - 40) + 0.1\,(40 - 5) + 0.5\,(5)]\,.7 = 6.3$$

(call 6 percent emulsified asphalt by weight of dry aggregate)

When MC or SC cutback asphalt is used, the asphalt content may be estimated by using the equation:

$$P = 0.02a + 0.07b + 0.15c + 0.20d \qquad (2)$$

where:

P = percent of asphalt by weight of dry aggregate,
a = percent of aggregate retained on the 300 μm (No. 50) sieve,
b = percent of aggregate passing 300 μm (No. 50) and retained on the 150 μm (No. 100) sieve,
c = percent of aggregate passing 150 μm (No. 100) and retained on the 75 μm (No. 200) sieve, and
d = percent of aggregate passing the 75 μm (No. 200) sieve.

Additional asphalt may be required for absorptive aggregates such as slag, limerock, etc.

Example:

Given the following aggregate and construction method determine the required cutback asphalt:

Method of construction: blade mixing;
Aggregate: relatively nonabsorptive gravel, gradation shown below; and
Asphalt: MC-250

Sieve size	19.0 mm (¾")	9.5 mm (⅜")	4.75 mm (No. 4)	2.36 mm (No. 8)	600 μm (No. 30)	300 μm (No. 50)	150 μm (No. 100)	75 μm (No. 200)
Percent passing	100	75	59	36	20	16	12	8

Solution:

$$P = 0.02\,(100 - 16) + 0.07\,(16 - 12) + 0.15\,(12 - 8) + 0.20\,(8) = 4.2$$

(call 4 percent cutback asphalt by weight of dry aggregate)

Types of Mixes

Open-Graded Mixes

A great variety of aggregate gradations have been used for open-graded cold mixes. Most of the research and development work on these mixes in recent years has centered in the northwestern states where they have been successfully used as bases and surface courses on many miles of federal, state and county highways and heavy-duty logging roads. Aggregate gradation and quality requirements vary somewhat but most of them closely approximate those listed in Table 13.2. Some attempts have been made to use aggregates with up to 20 percent passing the 2.36 mm (No. 8) sieve and 5 percent passing the 75 μm (No. 200) sieve and using CMS-2s* emulsified asphalt. Their performance as high-quality base and surface courses has been variable.

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*Some user agencies specify an additional cationic mixing grade designated CMS-2s. The CMS-2s is used for sand and silty sand mixes; it contains more solvent than standard CMS grades.

Table 13.2 Aggregates for Open-Graded Emulsion Mixes

	Base		Surface
Sieve Size	Coarse	Medium	Fine
38.1 mm (1½ in.)	100		
25.0 mm (1 in.)	95 - 100	100	
19.0 mm (¾ in.)		90 - 100	
12.5 mm (½ in.)	25 - 60		100
9.5 mm (⅜ in.)		20 - 55	85 - 100
4.75 mm (No. 4)	0 - 10	0 - 10	
2.36 mm (No. 8)	0 - 5	0 - 5	
1.18 mm (No. 16)			0 - 5
75 µm (No. 200)	0 - 2	0 - 2	0 - 2
Los Angeles Abrasion loss @ 500 Rev. (ASTM C 131)	40 max	40 max	40 max
Percent Crushed Faces	65 min	65 min	65 min
Emulsified Asphalt Grades	MS-2, MS-2h, HFMS-2, HFMS-2h, HFMS-2s, CMS-2 or CMS-2h		

Dense-Graded Mixes

Dense-graded aggregate mixtures are graded from the maximum size down to and including material passing the 75 µm (No. 200) sieve. They embrace a wide variety of aggregate types and gradations and, similarly, can be used for the full range of base and surface pavement types, depending on aggregate quality and equipment. Substantial savings can be realized when locally available, unprocessed aggregates are utilized on lightly traveled roads and in bases for heavy-duty pavements.

Samples of all aggregate intended for use in the mixture(s) should be submitted for testing. Recommended aggregate gradation and quality requirements for dense-graded mixes are shown in Table 13.3. For dense-graded mixes, recommended emulsified and cutback asphalts are shown on Table 13.1.

Sand Mixes

Except for aggregate gradation, the same basic principles apply for production of sand-emulsion plant mixes as for dense-graded, coarse aggregate emulsion mixes. Sand mixes may be used for either base or surface construction.

The addition of 1 to 2 percent portland cement will aid in the development of early initial strength. Thorough mixing is essential to uniform distribution of the cement throughout the mixture.

A wide variety of fine aggregates throughout the country has produced satisfactory results. Blending of two, or more, aggregates may be necessary to produce the desirable mix characteristics. The gradations in Table 13.4 have been used successfully.

Table 13.3 Aggregates for Emulsified Dense-Graded Asphalt Mixtures

Sieve Size USA Standard/Alternative		Semi-Processed Crusher, Pit or Bank Run	Processed Dense-Graded Asphalt Mixtures				
50 mm	(2 in.)	—	100	—	—	—	—
37.5 mm	(1½ in.)	100	90-100	100	—	—	—
25.0 mm	(1 in.)	80-100	—	90-100	100	—	—
19.0 mm	(¾ in.)	—	60-80	—	90-100	100	—
12.5 mm	(½ in.)	—	—	60-80	—	90-100	100
9.5 mm	(⅜ in.)	—	—	—	60-80	—	90-100
4.75 mm	(No. 4)	25-85	20-55	25-60	35-65	45-70	60-80
2.36 mm	(No. 8)	—	10-40	15-45	20-50	25-55	35-65
1.18 mm	(No. 16)	—	—	—	—	—	—
600 μm	(No. 30)	—	—	—	—	—	—
300 μm	(No. 50)	—	2-16	3-18	3-20	5-20	6-25
150 μm	(No. 100)	—	—	—	—	—	—
75 μm	(No. 200)	3-15	0-5	1-7	2-8	2-9	2-10
Sand Equivalent, Percent		30 min.	35 min.	35 min.	35 min.	35 min.	35 min.
Los Angeles Rattler @ 500 Revolutions		—	40 max.	40 max.	40 max.	40 max.	40 max.
Percent Crushed Faces		—	65 min.	65 min.	65 min.	65 min.	65 min.
Emulsified Asphalt		See ASTM D 977, D 2397/AASHTO M-140, M-208 or Specifications for Paving & Industrial Asphalts, SS-2, Asphalt Institute.					

The emulsified asphalt content normally varies within a range of 6 to 10 percent. Types SS-1, SS-1h, CSS-1, CSS-1h or HFMS-2s may be used along with a laboratory-determined amount of mixing water added to the sand.

Table 13.4 Sand Mixes

	Total Percent Passing		
Sieve Size	**Poorly-Graded**	**Well-Graded**	**Silty Sands**
12.5 mm (½ in.)	100	100	100
4.75 mm (No. 4)	75 - 100	75 - 100	75 - 100
300 μm (No. 50)	—	15 - 30	—
150 μm (No. 100)	—	—	15 - 65
75 μm (No. 200)	0 - 12	5 - 12	12 - 20
Sand Equivalent, percent	30 min.	30 min.	30 min.
Plasticity Index	NP	NP	NP

Construction Methods

Roadbed Preparation

The roadbed on which mixed material is placed must be shaped and compacted. The surface must be swept with a power broom to remove dirt and other foreign matter. Depending upon the condition or type of base or subgrade, a prime coat of MC-30 or MC-70 may be necessary. When applied the prime coat is allowed to cure, and after 24 hours any excess asphalt remaining on the surface is blotted with sand.

Windrows

In several types of road-mix construction, it is necessary to place the aggregate into windrows. Windrow sizers are used to ensure that the correct volume for the desired depth of pavement is placed.

Usually, loose material on the road surface is in too small quantity and is not uniform enough for use in the road mix. In such cases the material is bladed onto the shoulders. If it is practical to use the material from the roadway, it is bladed into a windrow and measured. Additional aggregate may be added as necessary to improve the gradation or to provide the required volume.

If two or more materials are combined on the area being surfaced, each is placed in its own sized windrow. These windrows are then thoroughly mixed before asphalt is added.

Before the correct amount of asphalt to be applied can be determined, it is necessary to determine the quantity of aggregate in the windrow. Measurements to be made on a sized or uniform windrow are shown in Figure 13.1. The quantities are then determined by the following formulas:

$$V = \frac{(A + B)\,C}{2} \tag{3}$$

where

V = volume of the windrow, square metre per linear metre (square foot per linear foot); and

A, B, C = dimensions of the windrow (Figure 13.1), metre (foot)

$$W_f = W_l\,V \tag{4}$$

where

W_f = quantity of aggregate, kg per linear metre (lb per linear ft.) of windrow; and

W_l = loose weight of dry aggregate, kg per cubic metre (lb per cubic ft.)

For example: a windrow of dried aggregate 0.5 m (1.64 ft) high, 0.5 m (1.64 ft) wide at the top, and 1.0 m (3.28 ft) wide at the base has a loose dry weight of 1500 kg per cubic metre (93.6 lb per cubic foot). Substituting into Eqs. 3 and 4:

Metric Units	Customary Units
$V = \frac{(0.5 + 1.0)\,0.5}{2}$	$V = \frac{(1.64 + 3.28)\,1.64}{2}$
V = 0.375 square metre per linear metre	V = 4.03 square feet per linear foot
W_f = 1500 (0.375)	W_f = 93.6 (4.03)
W_f = 562.5 kg per linear metre	W_f = 377 lb per linear foot

Adding Asphalt

Asphalt is either applied with a pressure distributor ahead of the mixing process or, in case of travel mixers, during the mixing process. In any case close control over the application rate and viscosity is necessary for proper

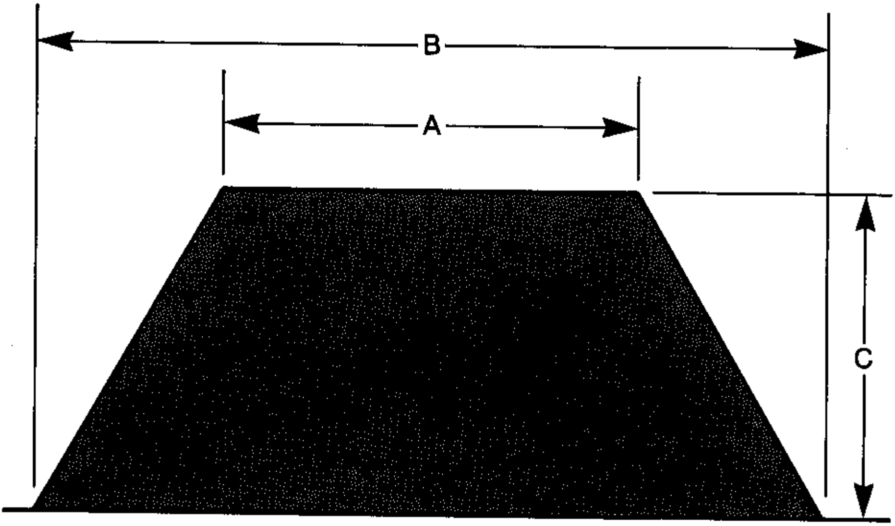

Figure 13.1 Measurements for Determining Windrow Quantities.

mixing. The rate of application for asphalt along the windrow is determined by:

$$A = \frac{W_f\, P}{100\, G} \tag{5}$$

where

A = application rate of asphalt, litre per linear metre (gallons per linear foot)
W_f = quantity of aggregate, kg per linear metre (lb per linear foot)
P = asphalt content, percent by weight of dry aggregate; and
G = weight of asphalt, approximately 1 kg/litre (8 lb/gal)

Example:

Aggregate: weighing 562.5 kg per linear metre (377 lb per linear foot) windrow

Asphalt: MC-250, asphalt content, 4 percent by weight of dry aggregate

Metric Units	Customary Units
$A = \frac{562.5\,(4)}{100\,(1)}$	$A = \frac{377\,(4)}{100\,(8)}$
A = 22.5 litres per linear metre	A = 1.9 gallons per linear foot

Proper viscosity is essential for the asphalt must be fluid enough to flow easily through the spray nozzles and to adequately coat the aggregate particles. Cutback asphalts, although already fluid, need some heating to bring them to the proper viscosity for spraying, which is 20 to 120 centistokes. But unless mixing begins immediately, the viscosity will quickly rise above that recommended for mixing, which is 150 to 300 centistokes. The volatiles in cutback asphalts keep them fluid long enough for the completion of road mixing, if it is done promptly. Even so, the aggregate temperature should be at least 10° C (50° F) in the shade at the time of mixing.

Mixing and spraying temperatures for emulsified and cutback asphalts are given in Table 13.5.

Table 13.5 Typical Asphalt Temperatures for Cold-Mixed Construction[1 & 2]

Type and Grade	Asphalt Temperature When the Asphalt is Metered through a Mixing Plant	Asphalt Temperature When the Asphalt is Applied to a Windrow Prior to Mixing
Emulsified Asphalts		
Anionic		
MS-1, MS-2, MS-2h HFMS-1, HFMS-2, HFMS-2h, HFMS-2s SS-1, SS-1h	10-70° C (50-160° F)	20-70° C (70-160° F)
Cationic		
CMS-2, CMS-2h CSS-1, CSS-1h	10-70° C (50-160° F)	20-70° C (70-160° F)
Cutback Asphalts		
MC, SC Asphalts		
250	55-80° C (135-175° F)	40° C + (105° F+)
800	75-100° C (165-210° F)	55° C + (135° F+)
3000	80-115° C (180-240° F)	——

[1] Temperatures are included in this table as a guide only.

[2] The maximum temperature of cutback asphalt shall be below that at which fogging occurs.

Methods of Preparing Mixes

There are four techniques for preparing paving mixtures for cold-laid pavements:

1. Blade mixing,
2. Rotary mixing,
3. Travel-plant mixing, and
4. Stationary mixing facilities.

All but stationary plant mixing are considered mixed-in-place techniques. The principal advantage of this type of construction is that it utilizes aggregate already on the site, or available from nearby sources. The aggregate and asphalt can be mixed in place with a minimum of equipment.

The in-place mixing systems provide low cost techniques for production of cold mixes. However, generally it is not possible to achieve 100% asphalt coating of the aggregate. Sufficient coating can be achieved to produce a tightly bonded mixture. The mixing procedure selected should be one that minimizes the amount of uncoated aggregate particles.

Stationary mixing facilities provide opportunities for better control of proportioning and mixing. In some cases, such as those mixes made with emulsified asphalt, no heating or drying of the aggregate prior to mixing is necessary.

Blade Mixing

Blade mixing is usually accomplished by the use of motor graders (Figure 13.2) or some type of multiple blade drag to blend the asphalt and the aggregate together. The aggregate is first air-dried by manipulating it across the roadway with the grader to expose it to the sun and air. After the moisture content has been reduced to 3 percent or less, the windrow is flattened or spread uniformly over about half the roadway.

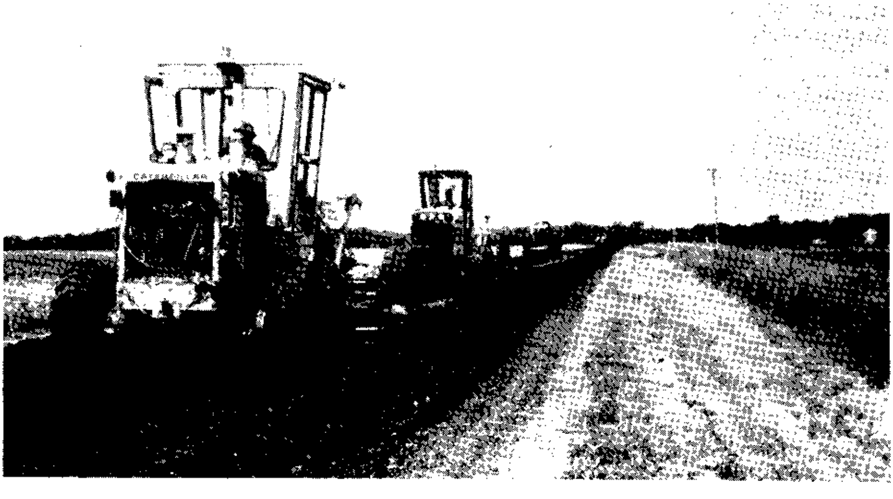

Figure 13.2 Blade Mixing.

Asphalt is then applied from an asphalt distributor in about two or three spraying passes. Immediately following each application, the asphalt and aggregate are partially mixed to leave as little free asphalt as possible on the surface and to prevent pools of asphalt from forming.

Partially mixed material, upon receiving the full predetermined amount of asphalt, is formed back into a windrow. It is then worked back and forth across the roadway with the blade until the aggregate particles are coated and the mix has a uniform appearance.

During mixing, attention should be paid to the position of the blade. It should be such that a complete rolling action of the material is obtained as the windrow is manipulated. Also during mixing, care must be taken not to incorporate extra material into the windrow and at the same time not to lose any of the material out of the windrow. After mixing and aeration have been completed, the windrow is manipulated to one side of the area to be surfaced in preparation for spreading.

Rotary Mixing

A rotary type mixer consists of a mobile mixing chamber mounted on a self-propelled machine. The chamber is open at the bottom and has widths varying from 1.8 to 3.0 metre (6 to 10 feet).

Inside the chamber there may be one or more transverse rotating drums that contain attached carbide cutting teeth. This type of equipment is particularly useful for mixed-in-place cold recycling where old hot or cold mix surfaces are to be incorporated into the mix.

The blades or cutting teeth serve a two-fold purpose, cutting the in-place material to a specified depth and mixing with asphalt. As the machine moves forward it strikes off the freshly mixed material to a predetermined level. The asphalt can be introduced either of two ways. In some mixers it comes through a spray bar that extends across the mixing chamber, the amount sprayed being governed by the forward speed of the unit (Figure 13.3). With the second method an asphalt distributor sprays the asphalt on the aggregate ahead of the mobile mixer. The materials are drawn into the mixing chamber as the machine moves forward.

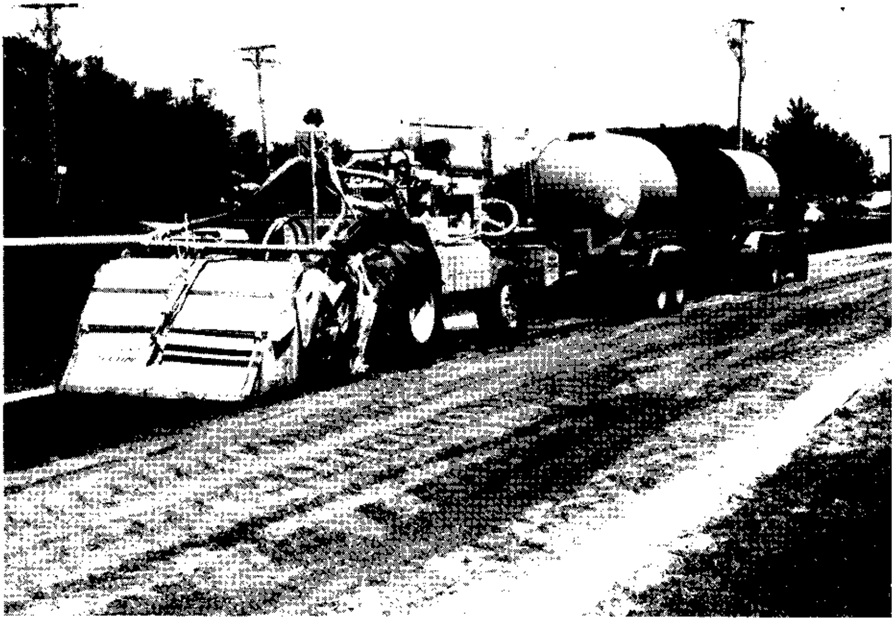

Figure 13.3 Rotary Mixer, Pulvimixer Type.

Travel-Plant Mixing

Travel plants are self-propelled pugmill plants that mix the aggregates with asphalt, applied at a controlled rate, as they move along the road. One type of travel plant has an aggregate receiving hopper, an asphalt storage tank, pugmill mixer and a floating screed. Either recycled material, virgin aggregate, or a blend of the two can be fed (by truck or low-lift loader from the windrow) into the plant for mixing with asphalt. The completed mixture is discharged at the rear and the thickness can be adjusted to any desired depth (Figure 13.4 and 13.5).

Travel plants should be capable of proportioning and thoroughly mixing asphalt and aggregates, uniformly dispersing the asphalt and adequately coating the aggregate particles.

The asphalt application rate for the windrow pick-up plant must be accurately matched with the width and thickness of the course, and the forward speed of the mixer. If all of the asphalt cannot be incorporated in one pass, the windrow should be split into two or more windrows and the asphalt added to each windrow for mixing. Depending on the type of travel plant and mix components, further mixing of the recycled material may be necessary with a motor grader. This ensures that all of the windrowed material is incorporated into the mix. It also aerates the mixture. The number of passes with the motor grader varies with job conditions.

Travel-plant mixing offers the advantage of achieving closer control of the mixing operation than is possible with blade mixing. The proportioning of the asphalt with the aggregate as well as the uniformity with which the two can be incorporated are of great importance.

The mixing machine moves along the windrow adding the asphalt and mixing it into the windrow as it moves forward. The rate of flow per minute at which the asphalt pump is adjusted may be determined by:

$$E = AS \tag{6}$$

where

E = discharge rate of asphalt, litres per minute (gallon per minute)
A = amount of asphalt, litres per linear metre (gallon per linear foot); and
S = forward travel speed, metres per minute (fpm).

For example: a windrow of aggregate is to have 20 litres (5.3 gal) of MC-250 asphalt per linear metre (3.28 per linear foot) mixed into it with a travel-plant mixer. For a forward speed of 5 metres per minute (16.4 fpm) the pumping rate for the asphalt is found by using Eq. (6):

Metric Units

$$E = 20\,(5) = 100 \text{ litres per min}$$

Customary Units

$$E = \frac{5.3}{3.28}\,(1.64) = 26.5 \text{ gal per min}$$

Should the windrow be too large for the travel plant, it may be split into two or more windrows and asphalt added and mixed to each separately. Further mixing of the windrow often is necessary and is usually accomplished with the motor grader. This also affords extra aeration of the mix.

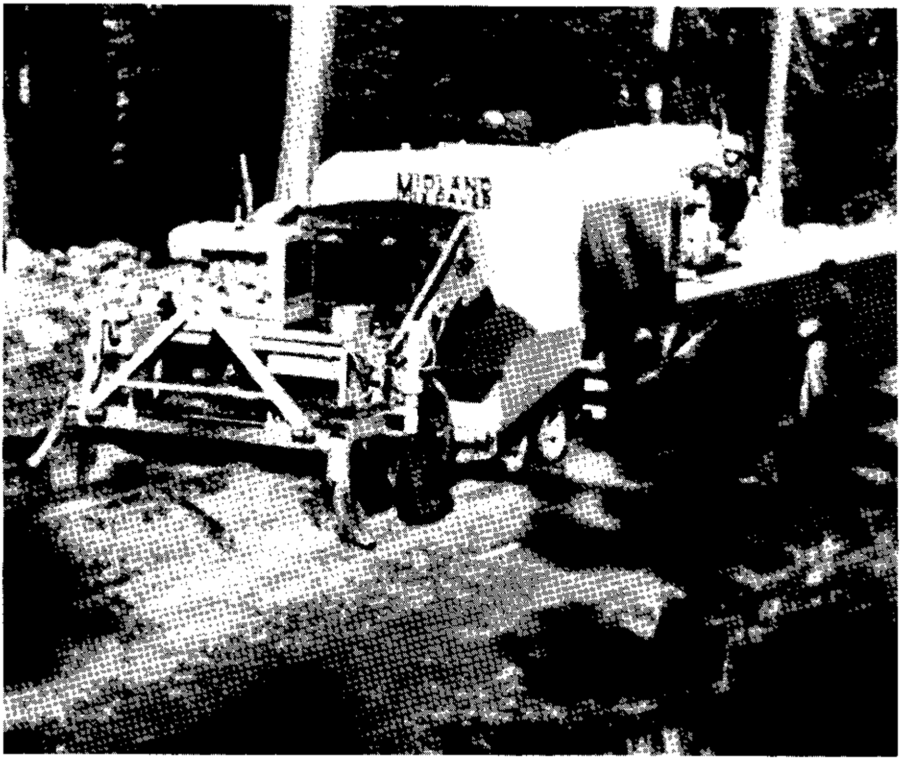

Figure 13.4 Travel Plant, Hopper Type.

Stationary Mixing Facilities

Mixes using emulsified and cutback asphalts prepared at a stationary mixing facility offer certain advantages over road-mixing methods. Weather conditions are not as much a factor in facility-mixing operations. Aggregates may be heated and dried just prior to mixing. Mixing is accomplished in a few minutes that would otherwise possibly require hours of windrow manipulation.

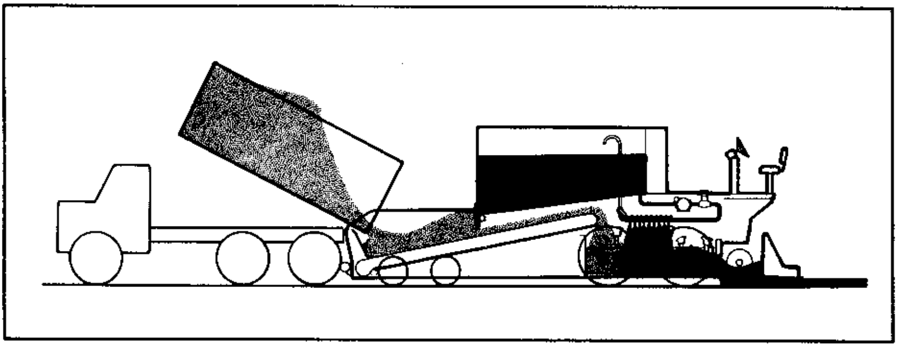

Figure 13.5 Travel Plant, Schematic.
(Courtesy Midland Machinery Co., Inc.).

The stationary mixing facility provides good control over both the proportions of materials in the mix and the mixing time. Uniformity of mixes is practically assured. Where the aggregates have been heated prior to mixing, a more viscous asphalt may be used. Often, enough aeration has been accomplished in the mixing operation—dumping and spreading mix in the roadway—to permit immediate compaction with rolling equipment.

Often, these advantages actually make facility-mixing methods more economical for producing and placing mixes in the roadway than road-mixing methods or procedures.

Cold-mix facility setups may vary depending on the quality and type of mix being produced. At the very least it should consist of a mixer, asphalt storage tank, asphalt metering pump, piping and spray bar equipment and spray bar for feeding water and additives, controls for adjusting and monitoring the various components, a conveyor for feeding the aggregate and, of course, a power source. It may also include one or more aggregate bins, proportioning aggregate feeders, scalping screen, aggregate load-sensing device, and surge bin or storage silo. Batch type pugmills can be used. However, this mix production is ideally suited for continuous mixers and these are used almost exclusively.

The production of high-quality cold mixes for heavy-duty pavements requires a well-controlled facility setup to ensure success. In addition to carefully monitored and controlled blending of aggregate, asphalt and, in some cases, water, the mixer should be of a type that permits variation in mixing time from 5 to 30 seconds. Surge hoppers or storage silos are highly desirable since they minimize facility shutdowns and improve mix uniformity. A typical cold mix facility is shown in Figure 13.6.

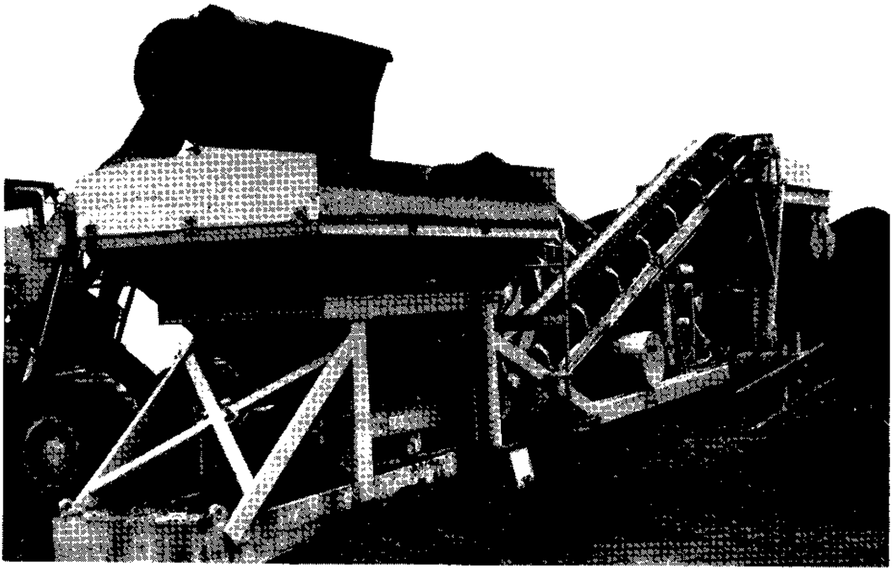

Figure 13.6 Cold-Mix Continuous Mixing Facility.

Aeration / Spreading / Compaction

Aeration of open-graded aggregate mixtures is usually not critical. However, as the presence of fine aggregate increases and the mix becomes dense-graded, ample aeration is necessary. Proper aeration for cutback asphalt usually is reached when the volatiles have been reduced about 50 percent of that contained in the original asphalt and the moisture content does not exceed 2 percent by weight of the total mixture. For emulsified asphalt mixtures the moisture content should be reduced to not more than 2 to 5 percent by weight of the total mix. In the case of sands and sandy soils that are used as a base course and surfaced within 30 days, volatiles in the mix should be reduced by at least two-thirds.

The point of adequate aeration is often made by experience and judgment. Experience with the particular mixture being prepared and trial rolling are of practical value for determining the presence of moisture or volatiles when test data are not immediately available. *Standard Method of Test for Moisture or Volatile Distillates in Bituminous Mixtures* (AASHTO T 110) can also be used. This test uses a metal still (similar to the one used for the distillation of emulsified asphalt), condenser, and a glass trap. A light solvent is added to a sample of the uncompacted mix that is in the still and then heated. Distillation may continue for as long as 1½ hours, which hardly makes the test an entirely satisfactory one for determining when sufficient mixing has been accomplished.

Mixtures that have been windrowed by a travel plant or by blade mixing are spread and shaped to the desired cross-section by successive passes with a motor grader. Mixtures blended by rotary cross-shaft mixers, although in place on the roadway, generally require additional shaping; this is also done with the motor grader.

Mixtures that do not require aeration may be spread to the required thickness immediately after mixing. Mixtures that require aeration are generally deposited upon the roadbed in a windrow. Because there is a tendency to leave a hump in the road when blade spreading from a centerline windrow, it is better to move the windrow to the side before spreading. Blade spreading should be accomplished in successive layers, with no layer thinner than twice the maximum particle size.

Construction should not continue during rainfall, or begin when rain is expected. Also, the emulsified asphalt base should not be placed if the ambient temperature is below 10° C (50° F). Emulsified asphalt mixes gain stability as the fluids (mainly water) evaporate. It is important not to retard this process. Therefore, lift thicknesses must be limited by the rate at which the mixtures loses its fluids. Factors affecting evaporation include the type of emulsified asphalt, the mix water content, the gradation and temperature of the aggregate, wind velocity, ambient temperature, and humidity.

Although each job has its own combination of these factors, experience has shown that under the best conditions, dense-graded mixes should be placed in compacted thicknesses no greater than 75 mm (3 in.). Open-graded mix may be placed in layers up to 150 mm (6 in.). When multiple lifts are required, some curing time must be allowed between successive lifts. The length of curing time varies with the rate of evaporation. However, a second lift can normally be placed after two to five days under good curing conditions.

A delay in initial rolling may be necessary especially with dense-graded mixes. Rolling seals the pavement as it reduces the voids in the mix. If done prematurely, it retards evaporation of the water in the mix and greatly extends the time required for the mix to reach required density. It also is necessary that the mix be allowed to develop sufficient cohesion to support the rollers. If it is delayed too long however, compaction will be difficult. In some cases, the developing asphalt-aggregate bond will be irreversibly broken; therefore, timing is important. Rollers may be selected as indicated in Table 13.6.

Because the tires of a motor grader compact the freshly-spread mix, the tracks will appear as ruts in the finished mat unless there is adequate rolling between each successive lift. Therefore, the roller should follow directly behind the motor grader in order to eliminate wheel tracks.

If the asphalt mixture exhibits undue rutting or shoving at any time during compaction, rolling should be stopped. Compaction should not be attempted until there is a reduction in moisture content, either naturally or by mechanical aeration.

After one lift is thoroughly compacted and cured, another lift may be placed on it. The operation should be repeated as necessary to bring the road to required grade and cross-section. For a smooth riding surface, the motor grader should be used to trim and level as the rollers complete compaction of the lift.

After the mat is shaped to the final cross-section, it is rolled with a steel-wheeled roller until the roller marks are eliminated.

Table 13.6 Types of Rollers Suitable for the Compaction of Cold-Mix Asphalt Bases

Type of Cold Mix Asphalt Base	Stage of Compaction: Breakdown Rolling	Stage of Compaction: Intermediate Rolling	Stage of Compaction: Finish Rolling
Dense-graded	Steel-wheeled Pneumatic-tired Vibratory	Steel-wheeled Pneumatic-tired Vibratory	Steel-wheeled
Open-graded	Steel-wheeled*	Steel-wheeled Vibratory Pneumatic-tired†	Steel-wheeled

*Breakdown rolling of open-grade emulsified asphalt base is often facilitated by adding a small amount of detergent to the sprinkler system water to prevent pickup.

†If intermediate rolling of an open-graded emulsified asphalt base is to be with a pneumatic-tired roller, or if traffic must use a recently-placed mix, a chokestone application of from 2.2 to 3.3kg/m^2 (4 to 6 lb/yd^2) of suitable aggregate should be applied. Stone screenings from a crushing operation or concrete sand are among those suitable.

13.2 Cold-Mix Recycling

In-Place Cold Recycling

General

Recycling by in-place construction procedures entails the following steps:

- Drainage
- Scarifying/Ripping
- Size Reduction
- Spreading
- Compaction
- Asphalt pavement course or surface treatment

Scarifying/ripping, size reduction, and mixing and spreading may be accomplished singularly or in various combinations, depending upon the equipment used.

Equipment for in-place recycling is essentially the same as that manufactured for conventional in-place construction. Since it is not feasible to include all variations of available equipment, this section covers only some of the equipment typically used in the recycling process.

Drainage

In the original design of a road, drainage is a most important consideration. It is equally important in rehabilitation work, such as recycling.

Large paved areas or real estate developments near the roadway may cause a runoff condition that will overload the roadway drains. The resulting subgrade saturation can cause pavement failures. Thus, a careful drainage evaluation should be undertaken as part of the design for the improvement.

The design and construction of subsurface drainage is discussed in Chapter 10.

Mix Design

For cold-mix recycling, the primary objective of the mix design is to produce a mixture comparable to one made from all new materials. However, there are no universally accepted mix-design methods for cold-mix recycling. In general, laboratory tests, empirical formulas or past experience with identical projects are used to establish the initial asphalt content, with the intention of adjusting it, if necessary, after construction begins.

The following are the steps in a mix-design procedure. For details on this procedure refer to *Asphalt Cold-Mix Recycling*, MS-21, Asphalt Institute.

With the information obtained from the materials evaluation, the recycled cold-mix design may be formulated. Figure 13.7 is a flow chart setting forth the steps for this interim design procedure.

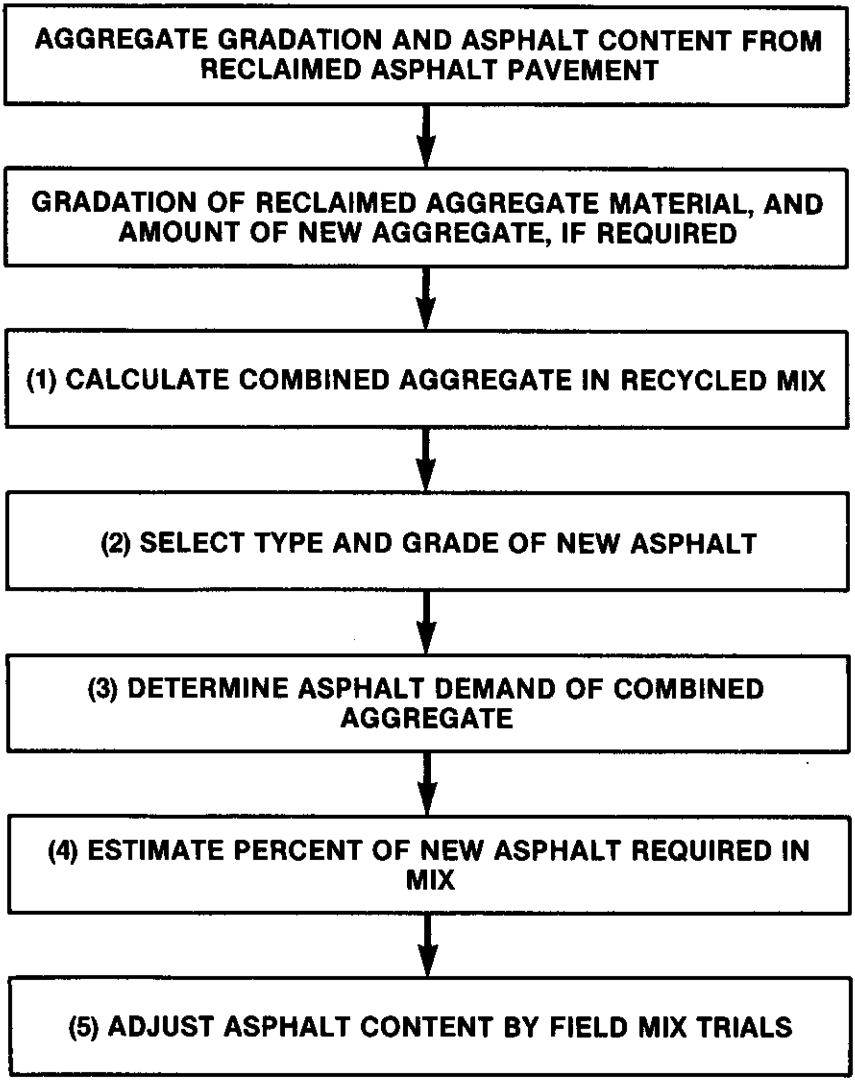
AGGREGATE GRADATION AND ASPHALT CONTENT FROM RECLAIMED ASPHALT PAVEMENT
GRADATION OF RECLAIMED AGGREGATE MATERIAL, AND AMOUNT OF NEW AGGREGATE, IF REQUIRED
(1) CALCULATE COMBINED AGGREGATE IN RECYCLED MIX
(2) SELECT TYPE AND GRADE OF NEW ASPHALT
(3) DETERMINE ASPHALT DEMAND OF COMBINED AGGREGATE
(4) ESTIMATE PERCENT OF NEW ASPHALT REQUIRED IN MIX
(5) ADJUST ASPHALT CONTENT BY FIELD MIX TRIALS

Figure 13.7 Flow Chart of Cold-Mix Design Procedure.

Scarifying/Ripping and Size Reduction

The first step in in-place recycling is to break up the road surface and/or base to the depth indicated by the structural design (See Chapter 11). There are two distinct methods employed: scarifying or ripping followed by size reduction and cold milling.

Scarifying/Ripping—This can be accomplished by:

(a) (Thin asphalt surfaces less than 50 mm [2 in.])—Motor graders with front-mounted scarifiers.

(b) (Surfaces 50-100 mm [2-4 in.] in thickness)—Motor Graders from 125-250 hp equipped with rear mounted ripper/scarifier. Ripping efficiency [(up to 150 mm (6 in.)] can be improved with radial type tires and ballasting.

(c) Thicker surfaces, in excess of 100 mm (4 in.) usually require track-type tractors with integral or pull type rippers.

Size Reduction—Alternative methods may be used as follows:

(a) The simplest is to use a cutter-crusher-compactor attached to a rear-mounted motor grader ripper-scarifier, combining the scarifying and size reduction operation. This attachment not only crushes thin pavements, but can also blend and aid in mixing the reclaimed material with the existing base course *(see* Figure 13.8).

(b) With front-mounted motor grader scarifiers, it is necessary to first windrow the material. Size reduction is then accomplished with a towed or self propelled rotary mixer or pulverizer (traveling hammermill, also called an impact breaker or preparator) *(see* Figure 13.9).

(c) Another type of rotary mixer that has been used to break asphalt pavement is a self-propelled multi-shaft, single-pass stabilizer. This type of machine has been employed as an integral removal, crushing and mixing tool. The asphalt bound surface must be thin enough to allow crushing and mixing by the unit in one pass *(see* Figure 13.10).

(d) For track-type tractor-ripped asphalt pavements, a towed grid roller, sheepsfoot or similar will break up the larger chunks for other pulverization methods *(see* Figure 13.11).

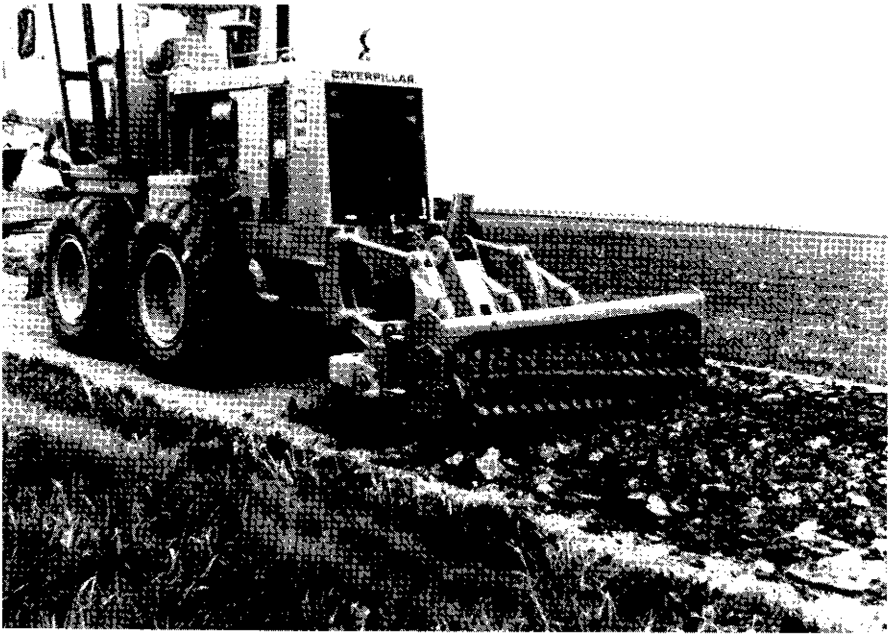

Figure 13.8 An Ateco Cutter-Crusher-Compactor Attached to a Motor Grader *(Photo Courtesy Caterpillar Tractor Company).*

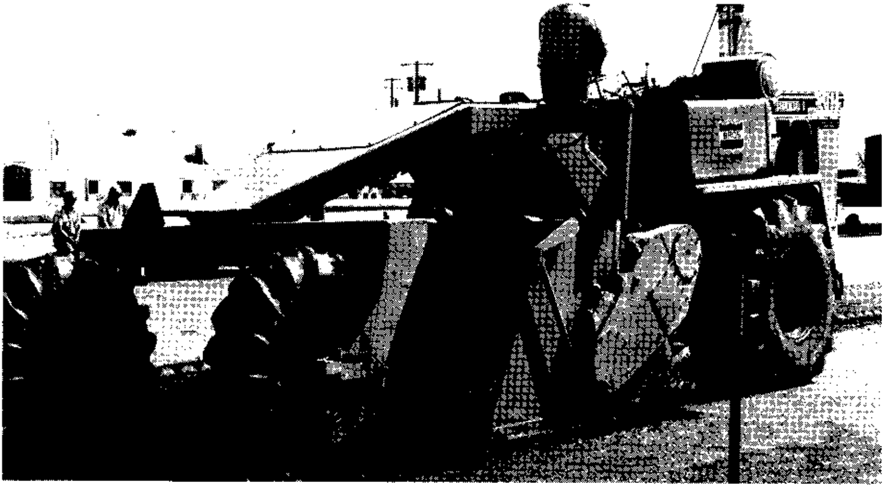

Figure 13.9 The Bros Reclaimer Size-Reduction Machine *(Photo Courtesy American Hoist & Derrick Company, Bros Road Machine Division).*

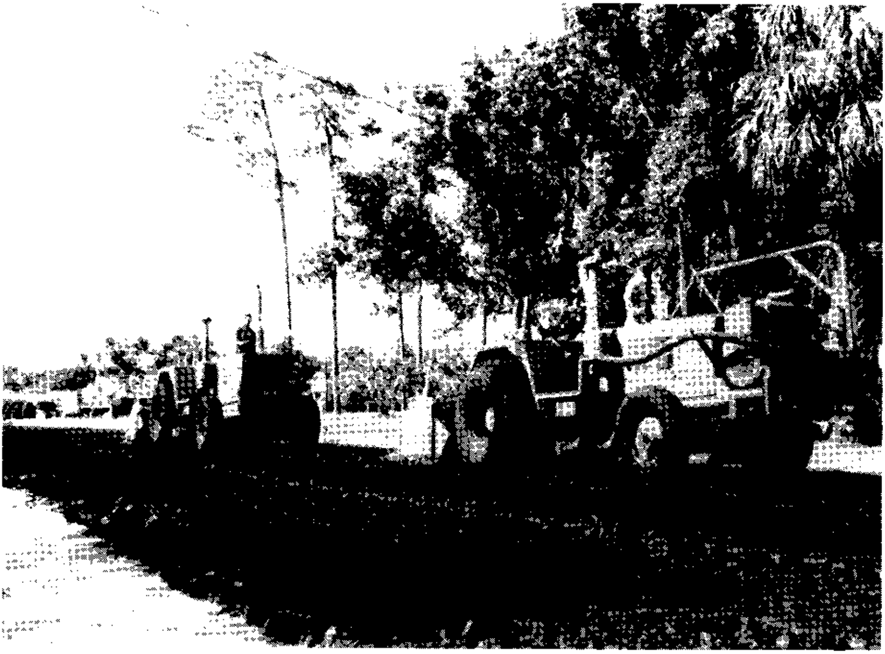

Figure 13.10 The Rexnord Pulvi-Mixer *(Photo Courtesy Rexnord Inc.).*

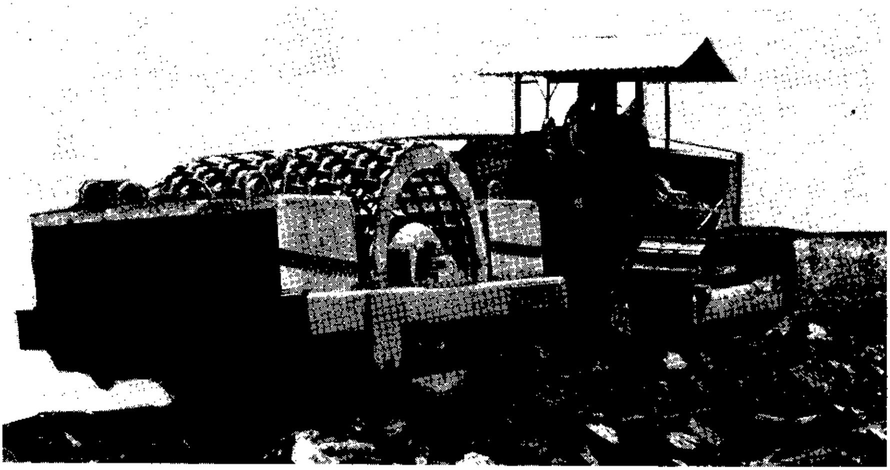

Figure 13.11 The Hyster Model D GRID Roller *(Photo Courtesy Hyster Company).*

Cold Milling—As an alternative to ripping and crushing equipment, cold milling machines may be used to directly reduce the asphalt pavement to the desired particle size (Figure 13.12). The depth of the cut, forward speed of the machine, rotational speed of the mandrel holding the cutting teeth, quality of the material, condition of pavement surface, ambient temperature and asphalt content determine the particle size produced.

Cold milling equipment has been used extensively for pavement removal and size reduction. Currently, most cold milling equipment are capable of cutting to a depth of 175 mm (7 in.) in a single pass.

Mixing

Mixing uniformly blends emulsified asphalt or hot asphalt cement with the sized asphalt pavement, old granular base and new aggregate, if used. When new aggregate is added to in-place materials, a spreader box or windrow sizer should be used to ensure the required quantity is evenly placed. Also, if cold-mixes are to be prepared by road mixing, accurately sized windrows are essential to ensure the required thickness of pavement and proper proportioning of the asphalt.

Depending on the equipment available, the type of asphalt used, and the degree of uniformity required, as discussed in Section 13.1, blade mixing, rotary mixing or travel plant mixing may be employed.

Spreading and compaction are equally similar as discussed in the previous section.

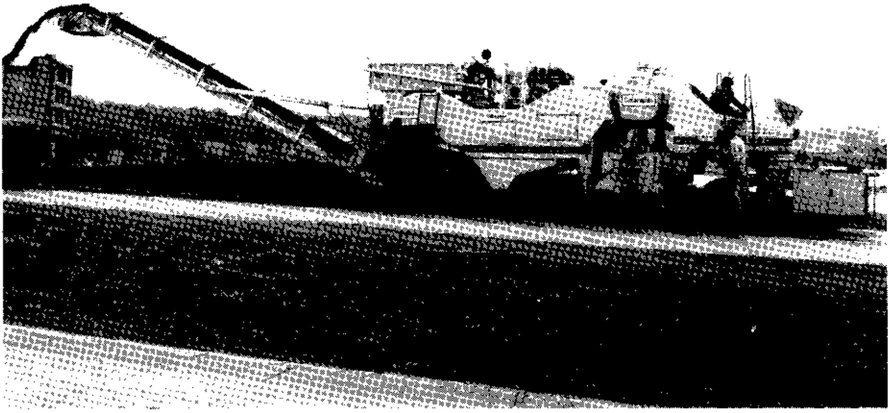

Figure 13.12 **The Cedarapids Cold Planer** *(Photo Courtesy Iowa Manufacturing Company, Cedar Rapids, Iowa).*

Central Facility Cold Recycling

General

Central facility mixing is used for asphalt cold-mix recycling projects that require high rates of production or close control of the mix design. Some of the equipment for in-place recycling is also used in conjunction with central facility equipment.

Reclaimed Materials

Reclaimed materials for central facility cold-mix recycling are obtained by ripping and crushing, or cold milling the old pavement. In the ripping and crushing operation, scarifiers or rippers are used to break up the asphalt pavement. Then it is loaded and hauled to a crushing and screening plant where the pieces of broken pavement are reduced to acceptable limits.

Specialized cold milling or planing machines have been developed that can reclaim asphalt pavement to controlled depths. In the process, the pavement is reduced to the desired maximum particle size; it is then hauled to the central facility. Untreated aggregate base to be processed is also transported to the central facility. It may be necessary to run this material over a scalping screen to remove oversize particles. Oversize material can cause placing (paving) problems and will produce a non-uniform ragged textured appearance to the pavement. The reclaimed mix, reclaimed aggregate and any new aggregate should be proportioned separately.

The height of reclaimed asphalt pavement stockpiles should be limited to prevent the crushed materials from sticking together because of dead load and high air temperatures. For the same reason, construction equipment should not be permitted on the stockpile. One way to minimize both sticking and excessive moisture is to coordinate the crushing and mixing operations so that a large stockpile of crushed reclaimed asphalt pavement is unnecessary. Reclaimed untreated aggregate should be stockpiled in the same manner as new aggregate.

Stationary Facilities

Central facility mixing is generally accomplished away from the road site. However, the equipment usually consists only of a mixer and certain auxiliary equipment for feeding the asphalt, water, reclaimed asphalt pavement, and aggregate to the mixer. It is therefore readily transported and easily erected. The asphalt and water pumps should be interlocked with the aggregate flow to produce a uniform mixture. A stationary facility may be either a batch, drum, or continuous type, although the latter is most often used for cold-mix recycling (Figure 13.13).

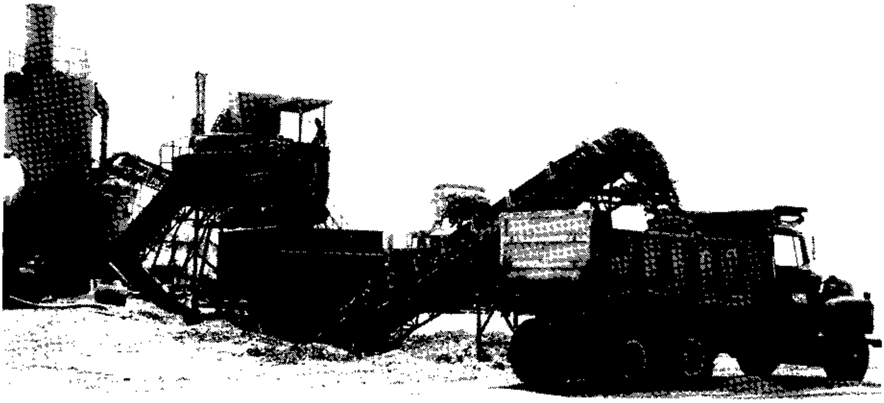

Figure 13.13 Cold-Mix Recycling Facility, Continuous Type.

Mix Preparation

In batch facilities, mixing is usually accomplished with a twin-shaft pugmill. Sized reclaimed asphalt pavement, reclaimed aggregate material and new aggregate (if required) are proportioned by weight with new asphalt and any needed mixing water before being fed into the pugmill. The batch is mixed and discharged into a haul truck before another batch is produced.

In continuous-mixing facilities, the devices feeding the materials are interlocked to automatically maintain the correct proportions. Typically, automatic feeders measure and govern the flow of materials in relation to the output of a positive displacement asphalt metering pump. Because asphalt is added to the mix on a volume basis, corrections must be made for the difference in volume according to temperature. As the proportioned materials move through the pugmill, the mixed material is discharged for subsequent hauling to the road site.

Mixing times can be varied in a continuous pugmill facility, by changing the arrangement of the paddles, by varying the height of the end gate, or by changing the location of the asphalt spray bar. With a drum-mix facility, mixing time is controlled by varying the slope of the drum or by changing the location of the asphalt inlet pipe wihtin the drum.

It should be noted that, with emulsified asphalt, regardless of the mixing method, 100 percent coating of the coarse aggregate particles is not always achieved, neither is it necessary. Additional coating will occur as the mixture is manipulated through the spreading procedure and during rolling. Some aggregate types might be difficult to coat properly, but this fact should be evident at the mix design stage. Mixing procedures should aim at achieving a uniform dispersion of the emulsified asphalt with a complete coating of the finer aggregate fractions.

Aerating Plant Mix

Mixtures that require aeration are generally deposited upon the roadbed in windrows.

Similar to mixed-in-place materials, central facility recycled cold-mixes gain stability as the water that has made the mix workable evaporates.

Spreading and Compacting

If mixing moisture can be controlled accurately to a level not requiring aeration or if climatic conditions and aggregate gradation permit evaporation of moisture without aeration by manipulation, a conventional self-propelled asphalt paver may be used to place recycled asphalt cold-mix.

The successful placement of cold-mixes with conventional pavers requires the presence of sufficient fluids. Dry mixes tend to tear beneath the screed or strike-off bar. If the mixture is too dry, the mix water content should be increased. When a self-propelled highway paver is used, heating the screed in attempts to eliminate this tearing does not help. It actually makes the mix less workable, since it serves to accelerate the drying process.

The mixture should be spread uniformly on the roadbed, beginning at the point farthest from the mixing plant. Hauling over freshly placed material should not be permitted except when required for completion of the work.

Spreading equipment such as Jersey spreaders and towed spreaders are also used for cold-mix recycling.

A Jersey spreader is a front-wheeled hopper that is attached to the front end of a crawler or rubber-tired tractor. The asphalt mixture is dumped into the hopper and the mixture falls directly to the road where it is spread and struck off to a controlled thickness. To begin spreading the mixture at the specified depth the tractor should be driven onto blocks or boards equal to the depth of the uncompacted spread and placed so that the tractor will ride directly onto the newly-placed material.

Towed-type spreaders are attached to the rear of haul trucks. The asphalt cold-mix is deposited into the hopper and falls directly to the surface being paved. As the truck moves forward, the mixture is struck off by a cutter bar, a blade, or by the screed.

The spreader should be towed at a uniform speed for any given setting of the screed or strike-off device. Variations in towing speed will vary spread thickness. Also, frequent stopping and starting may allow settlement of the screed and result in a bump in the pavement surface.

The compaction of recycled cold-mixes produced in central facilities is accomplished in the same manner as stated earlier in this Chapter for in-place cold-mix construction.

If traffic is to be permitted on a cold-mix surface immediately after compaction, it may be necessary to spread a light sand cover over the surface to prevent the mixtures from sticking to vehicle tires.

Chapter 14

Special Uses of Asphalt and Additives

There are occasions that call for unusual paving applications, such as the construction of asphalt athletic facilities or paving an embankment of a reservoir with an asphalt mix. Asphalt, as a cementing and waterproofing material, is a versatile material that can be used in many ways, some of which are described in this section.

The use of additives in asphalt or asphalt mixes to enhance one or more properties deserves some discussion. Based on the knowledge that additives are widely used in connection with other petroleum products such as motor fuels and lubricants, the Institute views with an open mind the idea of improving asphalts with additives to achieve certain engineering properties. There is some evidence based on field observations to indicate beneficial advantages of additives in pavement life. Special handling techniques are often required. Consequently, the Institute presents this material for information purposes but without recommendation for its use and suggests that cost effectiveness be based on field performance.

SECTION 14.1 Miscellaneous Asphalt Applications
☐ Bridge Deck Paving
☐ Roof-Deck Parking
☐ Industrial Floors
☐ Athletic and Recreational Facilities
☐ Hydraulic Structures
☐ Asphalt Roofs
☐ Asphalt in Railway Roadbeds
☐ Mulch Treatment

SECTION 14.2 Asphalt Modifiers
☐ Introduction
☐ Silicones
☐ Hydrated Lime
☐ Organic Anti-Strip Agents

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1. *Asphalt in Hydraulics, MS-12,* Asphalt Institute.
2. *Asphalt Pavement for Athletics and Recreation, IS-147,* Asphalt Institute.
3. *Asphalt in Railway Roadbeds, IS-137,* Asphalt Institute.
4. *Rail Transit Construction Using Hot-Mix Asphalt, TB-6,* Asphalt Institute.

14.1 Miscellaneous Asphalt Uses

There are some phases of asphalt construction that require special consideration of either the construction methods or the composition of the asphalt mixture. Asphalt as an engineering material is versatile and when designed properly is ideal for many forms of paving and other applications.

Bridge-Deck Paving

General

Asphalt surfaces are desirable for bridge decks because they are not affected by salts used for deicing, and if properly constructed they may waterproof the bridge floor, thus preventing water and salts from damaging it. Asphalt surfaces can also be highly skid-resistant, providing the safest bridge floor.

Asphalt Mixes

Dense-graded asphalt concrete mixtures are recommended for leveling and intermediate courses. In areas where chemicals are expected to be used to deice the pavement, aggregate that do not react with deicing chemicals should be selected. A lightweight aggregate may be desirable to reduce the weight of the asphalt pavement on the bridge.

Special asphalt mixtures have been used as bridge deck surfaces. One mix offers de-icing properties. It de-ices itself through the use of mix encapsulated calcium chloride pellets. These pellets break down under the action of traffic and form a mild brine which inhibits ice formation. Another mixture promises increased strength through the addition of polyester fibers. Still other rubber or elastomer modified asphalt mixes claim improved stability and fatigue life. All of these modifications are currently being used and evaluated on existing bridge decks.

Construction Methods

The surface to be paved, whether it is a portland cement concrete or a corrugated steel-deck floor, must be cleaned of all loose and foreign matter. Power brooms and compressed air tools may be necessary to accomplish this task. The surface of the pcc deck is then given a tack coat of SS-1h or CSS-1h asphalt emulsion diluted with an equal amount of water and applied at 0.2 to 0.9 litres/m^2 (0.05 to 0.20 gal per sq yd).

Steel decks after removal of mill scale usually require painting with a heavy asphalt paint. Vertical surfaces are also painted to a depth greater than the thickness of the subsequent paving. The tack coat is scrubbed in, thus providing a continuous asphalt film with no holes in the coating.

A leveling course of dense-graded asphalt concrete is laid and compacted. For steel decks, this course should have a compacted depth extending about

13 mm (½ in.) above the corrugations. Rolling the leveling course on a steel deck is done in a direction parallel to the corrugations.

The leveling course is then given a normal tack coat and the intermediate course (if required) is placed and compacted. This in turn is followed by a tack coat and the placing and compacting of the wearing course.

Pneumatic rollers having 550 kPa (80 psi) tire contact pressure are used for rolling all courses but finish rolling is typically done with steel wheeled roller. Inaccessible areas require hand or small vibratory compactors.

Roof-Deck Parking

General

The roofs of many modern structures serve as parking areas. Asphalt concrete paving is particularly adaptable in roof parking lots for reasons similar to those for bridge-deck paving.

The importance of proper drainage cannot be overemphasized. To ensure rapid runoff of water to drains the surface should be pitched or graded at a 2-percent slope or 20 mm/m (¼ in. per ft).

Asphalt pavement transfers large moving and static loads to the roofing membrane. Any lateral movement in the membrane causes the pavement to move. Any differential heaving under an asphalt pavement under load causes distress or failure to the underlying membrane. Also, any excess of free asphalt in the membrane is absorbed into the wearing surface and causes it to soften and possibly bleed.

If an asphalt pavement is placed over a deck area constructed of prestressed concrete beams, it is likely that reflection cracks might come through the pavement at the junction between successive beams. To minimize reflection cracking some type of stress absorbing membrane can be placed over each joint. This also helps to provide a tighter seal against the passage of water.

There are two types of roof-deck paving: (a) slab and pavement with a built-in movement plane, and (b) slab and pavement bonded together.

Free-Movement Plane Construction

Free-movement plane construction provides a distinct, free-movement plane between the structural roof slab and the pavement.

A positive bond breaker, such a lime or fine sand, is spread over the concrete deck to allow freedom of movement between the concrete deck and the pavement. A conventional built-up roof consisting of four or five plies of lightweight roofing felt is placed on the roof. However, the final flood coat and aggregate cover is not applied. In its place, a very light tack coat of diluted SS-1h or CSS-1h asphalt emulsion is applied to the top felt layer. On top of this, asphalt concrete paving is placed and compacted. A minimum thickness of 50 mm (2 in.) is recommended, and a very dense-graded mix having a maximum aggregate size of about 9.5 mm (⅜ in.) should be used.

Bonding Roof-Deck and Pavement

To bond pavement to a roof-deck after cracks and joints have been sealed, a conventional 4- to 5-ply built-up roof is applied. Each layer, except the surface of the top layer, is spot mopped with asphalt. No mopping is done within 400 mm (15 in.) of a portland cement concrete slab joint. The top layer is then covered with 50-75 mm (2-3 in.) of a dense asphalt concrete. Another method is to use an asphalt membrane that contains mineral filler and fibers for body and strength. The membrane is carefully spread to ensure a uniform thickness. Often, a protective board 3 to 6 mm (⅛ to ¼ in.) thick is placed over the membrane. The membrane or board is overlaid with one or two lifts of asphalt concrete having an asphalt content higher than normal. The total thickness of the mix should not exceed 75 mm (3 in.). The aim is to obtain an impervious mixture that weathers well and has a stability of more than 3336N (750 lb) Marshall.

Small rollers or sidewalk rollers are often used because of the restricted working area and the maximum allowable loads for the building. Small hand-operated electric or gasoline-powered vibratory compactors are used to work around areas inaccessible to rollers.

Industrial Floors

General

Asphalt floors are particularly adaptable for use in many factories and warehouses. Above ground level they are ordinarily laid on portland cement concrete decks. At or below ground level they may be designed and constructed similarly to highways or parking lots. Asphalt floors are waterproof, highly resistant to wear, and dustless. By selecting a highly siliceous mineral aggregate for use in the mix, they may be made resistant to certain dilution of acid.

Asphalt Mastic Floors

Originally, an asphalt mastic was prepared using crushed-rock asphalt. The term now generally applies to a mixture of graded sand, mineral filler, and asphalt cement quite similar in character to sheet-asphalt paving. Factory and warehouse floors are frequently subjected to very heavy unit loads, carried on small-diameter narrow metal wheels. When confronted by this condition, the regular sheet-asphalt mixture is modified by using a harder asphalt cement (AC-20, AR-8000 or 60-70 pen.) and a higher proportion of very fine mineral filler. A maximum thickness of less than 25 mm (1 in.) is typically recommended. Where acid resistance is not required, but where high stability and shock resistance are needed for forklift trucks, proprietary products of specially prepared emulsified asphalt, portland cement, plaster sand, and 9.5 mm (⅜ in.) rock are available.

Asphalt Concrete Floors

Dense-graded asphalt concrete such as used for paving highways and airports is particularly well adapted for paving warehouse and factory floors. Asphalt itself is unaffected by acids, but the mineral aggregate should be acid-resistant and care should taken to design and compact the mixture to produce high density and waterproofness.

The same basic principles should be used for the design of asphalt concrete warehouse floors as anticipated larger size aggregate mixes, such as those used in bases, have been used with a thin surface course or sealer to provide a smooth covering.

Athletic and Recreational Facilities

General

Almost every conceivable type of athletic or recreational facility can use asphalt in some way—from running tracks to auto race tracks, (Figure 14.1), to playgrounds to football fields (Figure 14.2). A wide selection of all-weather asphalt construction is economically available for these purposes. When properly designed and constructed, maintenance costs are negligible.

Construction

Drainage, of course, is the first consideration. With full-depth asphalt construction, only surface draining is usually necessary. Either hot- or cold-laid mixtures can be used in most instances. Construction procedures used in road building can generally be followed in paving athletic and recreational facilities with close attention given to maintaining proper grade.

Many proprietary products have been developed and have become accepted for play areas and game courts. Some of these are synthetic surfaces such as artificial grass placed on an asphalt base. Others have emulsified asphalt combined with selected pigments and fillers for squeegee application on an asphalt-concrete surface. These materials develop a tight seal with no loose material remaining. They present smooth resilient surfaces that are easy to clean and maintain.

Full-depth asphalt construction has contributed greatly to the use of synthetic surfaces and artificial turfs on playing fields. The asphalt base provides a true-to-grade surface for the rubber or cushion layer that is bonded to and sandwiched between the asphalt and artificial grass mat.

To obtain true-to-grade surfaces, the asphalt is usually placed in at least two courses. The top layer is usually an asphalt-concrete surface course, sand, or sheet asphalt.

Figure 14.1 Asphalt Concrete Surfaced Race Track.

Hydraulic Structures

General

A great deal of experience has been gained in the application of asphalt to hydraulic structures of all kinds. Asphalt linings may be selected on the basis of their intended function. These are shown in the following chart:

Asphalt Lining	Function Waterproofing	 Protective Cover
Sprayed asphalt membrane	×	
Dense-graded hot-mix asphalt	×	×
Open-graded hot-mix asphalt		×
Prefabricated linings	×	×

Sprayed Asphalt Membrane Linings

An asphalt membrane lining consists of a continuous layer of asphalt cement sprayed over the surface being waterproofed. It is usually covered with a layer of soil, gravel, or other material to hold the lining in place, to prevent weathering (oxidation) of the asphalt, and to protect it from mechanical damage. The asphalt is usually sprayed in two applications at a total rate of about 6.8 litres/m^2 (1½ gal per sq yd) to obtain a layer thickness of about 6 mm (¼ in.).

Special asphalts have been developed for this type application. There are specially blown asphalt cements having a high softening point of 79-93° C

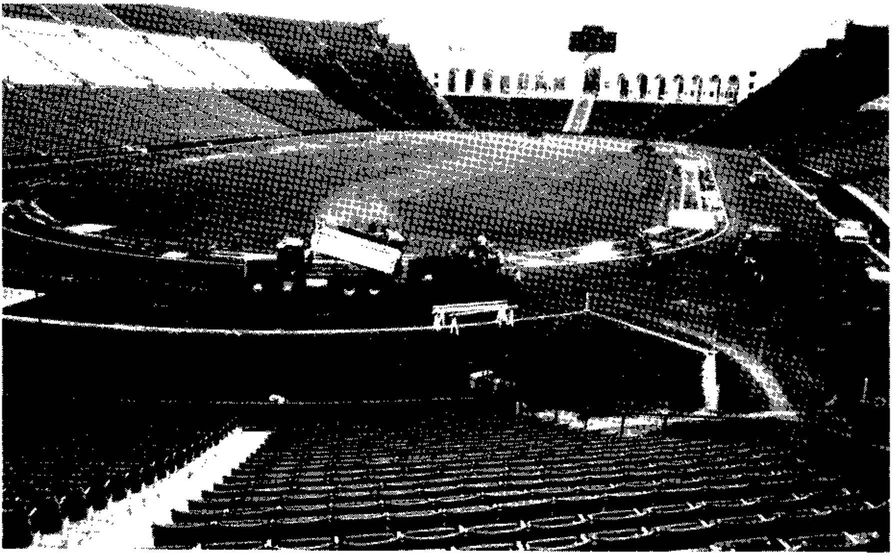

Figure 14.2 Los Angeles Stadium.

(175-200° F) and possibly a ductility as low as 3.5 cm.

The subgrade should first be sterilized to deter weed growth. The surface should be free of lumps, clods, and large rocks. It is rolled lightly before asphalt is applied. Application temperatures should be from 177 to 218° C (350 to 425° F). Cover material is carefully placed so as not to puncture the membrane. The protective cover is usually at least 0.3 m (1 ft.) thick.

Although developed as a canal lining, there has been extensive use of the asphalt membrane in all kinds of reservoirs (e.g., water supply, sewage lagoons, brine pits, fish ponds, and farm ponds).

Dense-Graded Asphalt Concrete Linings

Dense-graded asphalt concrete is especially well adapted to the construction of linings that are watertight and erosion resistant. Mix designs for this purpose are very similar to dense-graded asphalt mixes for highway pavements, but should limit design compaction by the Marshall Method to 35 blows. They differ by having a higher mineral filler and asphalt content than highway mixes. Reasons for this are basic. The material will probably not get the degree of compaction, especially on the slopes, that highway pavements receive. The extra asphalt and mineral filler help in obtaining a voidless mix. Extra asphalt also increases the durability of the mixture and provides a thicker film coating around the aggregate particles. Harder asphalt cements (AC-20, AR-8000, or 60-70 pen.) are usually preferred for hydraulic mixes.

High stability is not necessary. If a mix can be satisfactorily placed and compacted on a slope at high temperatures, it usually has adequate stability.

Impermeability of a dense-graded asphalt concrete lining is usually ensured when voids in the mix are 4 percent or less and the lining is at least 50 mm (2 in.) thick. These mixers will typically have a coefficient of permeability "k" of less than 1×10^{-7} cm/sec., which is generally considered impermeable to water.

Asphalt mixes are placed on slopes usually by conventional spreading machines that are raised or lowered on the slopes. Spreaders and other equipment are attached to cables and winched up and down the slope from cranes or bulldozers on the crest. On some of the larger projects, finishing machines and rolling equipment have been modified for slope paving. Modification is usually left to the imagination and ingenuity of the contractor because standard machines are not produced specifically for this purpose.

Asphalt paving mixtures are used for the construction of various types of containment basins—water treatment and storage or other types of chemical treatment basins. It cannot be used if the chemical treatment involves a petroleum solvent since this could cause a softening or dissolving of the asphalt cement binder. Asphalt/aggregate mixtures, however, are highly resistant to acids and alkalies but the aggregates involved should be checked for assurance that they are not reactive with the material being stored or treated. Because asphalt is an inert, non-contaminating material, it is an ideal material for storage and/or treatment of potable water. When used as a containment basin, a voidless hydraulic mix should be employed. Details of design and construction are contained in the Institute's Asphalt in Hydraulics, MS-12.

Open-Graded Asphalt Linings

Open-graded or porous asphalt mixes have a greatly reduced sand content and are practically devoid of mineral filler. As a consequence, the asphalt content is also reduced. The purpose of this type of lining is to provide an erosion-resistant cover to an earth lining, such as clay, that provides watertightness for the structure. The porosity of the mix allows free passage of water to and from the earth blanket with no danger of hydrostatic pressure under the asphalt layer. The required thickness of these linings is governed only by the need for them to remain intact under the erosive forces of water such as wave action. Typical thickness is about 75 mm (3 in.).

Porous mixes may also be used as a supporting layer for a watertight lining. This combination adds strength to the lining of the structure and at the same time provides a drainage layer to prevent the buildup of hydrostatic pressure by the lining that could rupture or damage the watertight layer.

Prefabricated Asphalt Linings

Prefabricated asphalt panels are usually 0.9 or 1.2 m (3 or 4 ft) wide and have lengths of 3.7 m (12 ft) or more. Thicknesses may vary from 3 to 13 mm (⅛ through ½ in.). The 13 mm (½ in.) panel has been the most commonly used.

The core is usually a dense mixture of asphalt and filler. Outside layers are made of some tough, asphalt-impregnated material, such as fiberglass, or one of the synthetic fibers or plastics. Then the panel is molded under heat and pressure.

Constructing linings from prefabricated materials does not require heavy paving machinery. Prefabricated materials may be used where space limitations prevent the use of machinery. They are used to reline cracked and deteriorated concrete linings (as a lining itself), or where a lining must conform to strict thickness specifications.

Open-Graded Asphalt Drainage Courses

Environmental restrictions will sometimes prohibit the flow of surface runoff from paved areas into surrounding drainage facilities and streams. This restriction often is to prevent overcharging these systems. As a result, open graded asphalt drainage courses have been developed. These porous pavements permit water to rapidly drain through the pavement and into the underlying substructure. This type of pavement should be used only when appropriate subsurface drainage capabilities are present. The pavement is usually composed of a predominately one-sized aggregate with relatively lower asphalt content. Voids in these pavements range from 15 to 25 percent. Periodic vacuuming of the pavement may be necessary to remove soil particles and maintain the pavements drainage capability.

Asphalt Roofs

History

Roofing was probably the world's first craft, since shelter has always been a primary material need of man. From the beginning, he has had to protect himself against the relentless forces of nature: from extreme heat and cold, from rain, wind, hail, snow, and ice.

Asphalt, with its extraordinary combination of waterproofing, preservative, and cementing qualities, was an early discovery of ancient peoples. They used it extensively in the construction of their buildings and roads, many of which, after thousands of years of exposure to the elements, are still existing, well preserved.

Today, asphalt is recognized more than ever as the outstanding roofing material and it is used extensively to cover all kinds of buildings, from homes to factories. Asphalt roofing materials and construction methods are fully covered by specifications of the American Society for Testing and Materials and many government and private agencies. In addition, all types of asphalt roofing are sanctioned by federal and other public agencies.

Asphalt roofs may be built in single or multiple-layered systems with impregnated fabrics or with manufactured shingles.

Types of Asphalt Roofing

Asphalt roofing is of two general types:

(1) Prepared Roofing

a. Asphalt shingles. This type is composed of three basic materials—

asphalt, felt, and mineral granules. The felt is impregnated with an asphalt saturant. Both sides of the saturated felt are then covered with a harder, tougher coat of asphalt, in which are embedded mineral granules. This composition is then cut into individual shingles or strips. Asphalt shingles are available in almost any color from pastels to deep rich tones. Factory techniques of application of granules have improved with the development of new permanent colors, and granules may be applied in almost any blend or pattern of colors desired.

b. Asphalt roll roofing—smooth surfaced. This type is made up of a single layer of roofing felt saturated with asphalt and coated on both sides with a harder asphalt.

c. Asphalt roll roofing—mineral surfaced. This type consists of smooth surfaced roll roofing in which mineral granules have been embedded on either one or both sides.

(2) Asphalt built-up roofing. An asphalt built-up roof is composed of several layers of asphalt-saturated felt applied principally to a flat roof-deck. Asphalt is applied between the felt layers for adhesion. Over the top layer a flood coat of asphalt is then added and this is covered with gravel or slag.

Prepared roofings are those products which are manufactured and packaged ready to apply to the roof deck, usually by nailing only. Asphalt shingles and certain roll roofings are in this category. They consist of a base of saturated felt to which has been added an asphalt coating and, usually, a surfacing of mineral granules.

Of the types listed the shingle roof has been the most common. Asphalt shingles are applied to the steep roofs of homes, barns, and similar structures. They fill the need for an attractive, efficient, and fire-resistant but low-cost roofing for residential construction.

The built-up roof is a contrasting type, constructed on the flat or nearly flat tops of very large buildings—industrial, public, and many others. These roofs are literally paved with asphalt, although in a manner quite different from the paving of roads.

For built-up roofing, only the components are manufactured and delivered to the job. The roofing itself is built up directly on the roof deck in a series of layers or plies, sandwiched between layers of asphalt. A cover of aggregate is then applied over the final layer. The mopping on of the asphalt and the spreading of slag or gravel may be accomplished with the most readily available and simplest type of equipment. All operations can be performed quickly by the hand method of application, although for very large roofs mechanical felt-laying machines, hot-asphalt applicators, and gravel spreaders will do the job more easily and rapidly.

The heart of the entire built-up roofing operation is the kettle for heating the asphalt to the proper temperature for application. For efficient operation the kettle must be able to bring the asphalt up to this temperature in a short period of time without overheating at the start, or at any time during the kettle's operation. Overheating may seriously damage the asphalt and considerably reduce the life expectancy of the roof. Overheating may also cause the asphalt to be applied too thinly, resulting in poor adhesion between the roofing felts and possible later separation of the plies.

Properly constructed, an asphalt built-up roof will remain in service condition for many years with little or no maintenance required.

There are many roofs on which either prepared or built-up roofing can be used, but the choice is limited somewhat by the slope of the deck to which it is to be applied. Built-up roofs are usually applied only on the flatter slopes while the application of asphalt shingles and other prepared roofing is usually confined to the steeper slopes.

Asphalt in Railway Roadbeds

Asphalt's advantages for railroad roadbeds lie in its unique qualities of strength, resilience and waterproofness; and in the fact that asphalt pavement can be constructed quickly using equipment already available in highway building.

Asphalt's properties render it the ideal material for today's special-car, heavy wheel-load freight trains, and for tomorrow's very high-speed passenger services.

Asphalt provides two options for roadbeds. The first is *Full-Depth asphalt* construction in which a scientifically-designed heavy-duty layer of asphalt concrete is placed directly on the subgrade. The rails and ties are placed directly on the asphalt layer without intervening ballast. The second is designated *underlayment* construction. In this a thinner layer is constructed on the subgrade, with ballast being placed on top of the layer and under the ties.

Figure 14.3 shows a conventional ballast section and the two asphalt options. Both the Full-Depth and underlayment sections can be built at reasonable cost. However, the underlayment system is more readily adapted to present-day railroad maintenance practices. As advances are made in maintenance and construction techniques, both methods should be found to be both acceptable and advantageous.

An unprotected subgrade requires a much shorter maintenance cycle than one underneath an asphalt course. Lack of this kind of protection means frequently added ballast and track realignment.

Asphalt courses, either Full-Depth or underlayment, protect the subgrade from water saturation and consequent loss of vertical or horizontal gradeline.

Asphalt has great potential for the following types of application:

(1) New construction and/or rehabilitation of freight and passenger lines and yards.

(2) Extending passing tracks and double tracking.

(3) Turnouts, crossovers and crossings (rail and highway).

(4) Upgrading and correction of in-service track exhibiting poor soil and bad drainage pumping conditions.

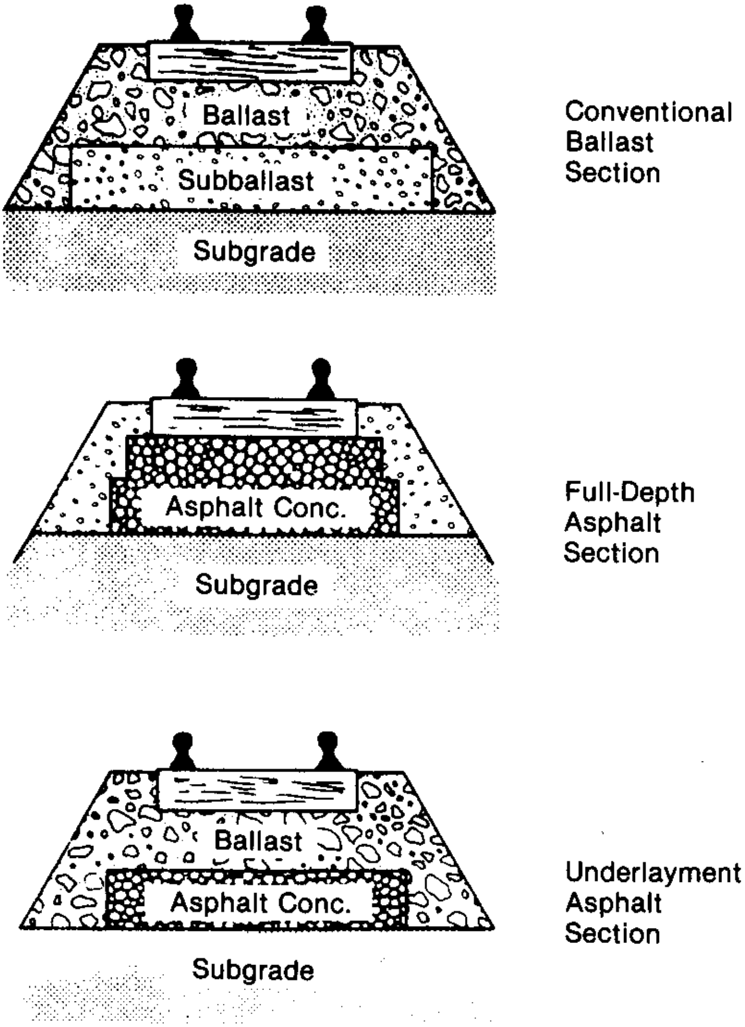
Ballast
Subballast
Subgrade
Conventional Ballast Section
Asphalt Conc.
Subgrade
Full-Depth Asphalt Section
Ballast
Asphalt Conc.
Subgrade
Underlayment Asphalt Section

Figure 14.3 Asphalt Railway Roadbed Cross-Sections.

(5) Transitioning bridge approaches and other rigid track sections.

Among the several advantages of asphalt railroad roadbeds are the following:

- They provide increased load-carrying capabilities though improved support for the track structure.
- Maintain line and surface within required tolerances for much longer than is the case with conventional sections, thus reducing time and money expended on maintenance.
- Provide an impermeable barrier that prevents surface water from softening the soil subgrade.

- Prevent subgrade soil from intruding into the ballast (pumping), and halt the loss of ballast through it being driven down into the subgrade. Also minimize ballast degradation by means of improved stability.
- Facilitate better and faster train schedules through minimizing delays caused by track maintenance.
- Improve passenger riding comfort and eliminate freight damage by means of improved line and surface.
- Allow the use of lower-cost, locally produced aggregate.
- Reduce track maintenance in switching yards and thus minimize traffic disruption.
- Reduce the thickness of conventional processed aggregate ballast layers.

For design information on asphalt railroad trackbeds, refer to Hot-Mix Asphalt Railroad Trackbed: Construction and Performance, RR-83-2, Asphalt Institute.

Mulch Treatment

General

Soil erosion caused by water and wind can present a serious problem in the construction of embankments and flat areas adjacent to highways. The most common method of combating this problem is the use of vegetation to stabilize these areas. But, during the period between when the seeds are planted and germination takes place they are susceptible to being blown or washed away. Several procedures have been developed to protect the planting until the seeds germinate and a root system forms. One of the most effective is the use of emulsified asphalt. It leaves a thin membrane over the seeded area or holds a hay or straw mulch in place. Both approaches have been used successfully. Because they differ in procedure each will be discussed separately, although both are designed to achieve the same result.

Emulsified Asphalt Spray Mulch

In this system the asphalt emulsion is sprayed directly onto the seeded area, forming a thin membrane cover. The thin film of asphalt has three beneficial effects:

1. The asphalt cover holds the seeds in place and prevents their loss by the eroding forces of wind and water.
2. Because of its dark color, the asphalt absorbs and holds solar heat during the germination period.
3. The asphalt membrane tends to hold moisture in the soil, thereby promoting faster plant growth.

As the young seedlings emerge from the soil they can easily break through the thin asphalt cover. The membrane eventually disintegrates as the seedlings mature and cover the ground area.

Emulsion grades commonly used in this operation are SS-1, SS-1h, CSS-1, or CSS-1h. It is normally applied at a rate of 0.70 to 1.35 litre/m^2

(0.15 to 0.30 gal/yd^2). The exact amount is determined by the nature of the soil and the slope of the area being treated. Special care must be taken to apply the optimum amount of emulsified asphalt. Too little may not hold the soil against erosion by wind and water. Too much emulsion may leave a thick membrane, which would delay growth. The area that is to receive the emulsion spray must be reasonably smooth so that a uniform coating can be applied. Depressions in the surface may collect pools of asphalt and ridges may be coated on one side with virtually no asphalt on the other.

The emulsion can be applied with a hand-held spray nozzle or with an offset distributor bar attached to an asphalt distributor truck.

Emulsified Asphalt Mulch Tie-Down

Asphalt emulsion can be used for anchoring straw or hay to a seeded area. There are two approaches that can be used.

In one case the straw or hay mulch is distributed over the prepared area at a rate of 3.3 to 4.5 tonnes/hectare (1½ to 2 tons/acre). The seed is then mixed with water and liquid fertilizer and applied with a hydraulic seeder. A spray application of 0.45 litre/m^2 (0.10 gal/yd^2) of asphalt emulsion follows. The emulsion can be applied in a solid pattern or a saw-tooth, checkerboard, or perpendicular line pattern. The solid pattern is most effective, especially when the wind velocity is high. If the amount of mulch is increased above 3.3 to 4.5 tonnes/hectare (½ to 2 tons/acre), the amount of emulsified asphalt applied must be increased proportionately.

A second method begins with the hydraulic application of seed and fertilizer directly to the prepared soil. Then, the mulch and emulsified asphalt are ejected at the same time through a special blower equipped with twin jets, (see completed section, Figure 14.4). The two materials are mixed in flight. This is the preferred method as it has at least two advantages:

- The mulch and the asphalt emulsion are applied in a single application, which reduces costs and reduces the time required.
- It results in better bonding between the emulsion and the hay/straw mulch.

The same types of asphalt emulsion can be used as recommended for the emulsified asphalt spray mulch.

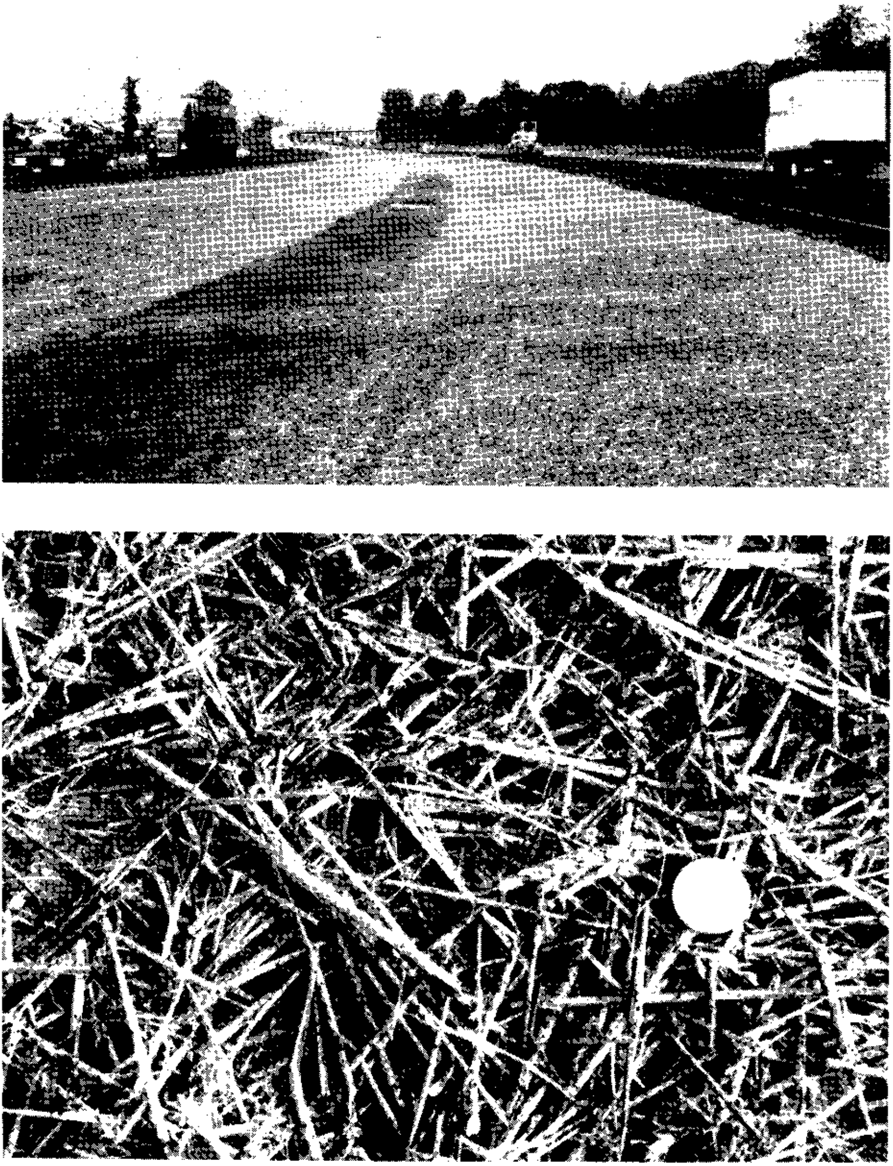

Figure 14.4 **Completed Section of Interstate With Emulsified Asphalt Mulch on Median and Close-Up.**
(Courtesy of Ohio Department of Transportation).

14.2 Asphalt Modifiers

Introduction

Asphalt modifiers are materials incorporated in an asphalt paving mixture in addition to the normal asphalt, aggregate and mineral filler contents. Modifiers are generally added with the intention of controlling pavement rutting; pavement cracking; asphalt oxidation; or pavement water damage.

Commercially-available asphalt modifiers may be grouped for convenience into the following major types:

- Polymers, including elastomers
- Metal complexes
- Elemental sulfur
- Fibers
- Hydrated lime and portland cement
- Silicones
- Fillers, and
- Organic anti-strip agents

Many of these materials, although commercially available, have not been used extensively in routine highway construction and rehabilitation. However, polymer modified asphalt crack sealers used in maintenance operations are gaining in popularity. As is true for most new highway materials or techniques, many years of service may be required to prove the utility of an asphalt modifier in achieving the intended effect on asphalt pavement performance.

Generally, the use of an asphalt modifier increases the first cost of an asphalt pavement compared to that of one constructed conventionally. While laboratory testing may indicate significant benefits from the use of the modifier, its adoption into routine operations may be hampered by a lack of corroborating field performance data showing that the higher first cost is balanced by extended service life.

Three types of modifiers are used routinely: silicones, organic anti-strip agents and hydrated lime; these will be discussed in some detail.

Silicones

Silicones are added to asphalt under certain conditions to reduce or nullify the effects of moisture or other conditions. For example, silicones may be added to hot asphalt to prevent foaming while in hot storage. Silicones in asphalt mixtures have been employed to reduce hardening of asphalt concrete in storage, to prevent slumping and segregation of the mixture during transportation, and tearing of the mix during laydown.

Silicones are added to asphalt in concentrations of 1 to 3 ppm. For quick blending of silicone in asphalt, it is sometimes diluted with diesel oil. When this is done, the amount of diesel oil must be minimized to avoid changing the properties of the asphalt, particularly its consistency or viscosity.

Silicones are very persistent materials. Their effect may be retained or carried over in the tank from filling to filling to the extent that some test results are rendered unreliable.

Organic Anti-Strip Agents

Organic anti-strip agents in asphalt are used to promote coating and adhesion, increasing the resistance of the asphalt film to stripping from aggregate surfaces by the action of water.

To date, there have been no field performance data published or otherwise disseminated showing conclusively that organic anti-strip agents are economically desirable or yield longer asphalt pavement life. There is some evidence, based on field observations, to indicate that their use under adverse conditions may sometimes be beneficial in the early stages of pavement life. How long these benefits last or whether the agent contributes to a longer service life of the pavement is presently unclear.

Empirical laboratory tests and arbitrary specification limits do not provide conclusive engineering basis for using organic anti-strip agents. In most instances, the correlation of these tests and specification limits with field performance is not established. Even where some degree of correlation does exist, there is a significant imprecision in the prediction of field performance from laboratory results.

Hydrated Lime

Hydrated lime (and to a lesser extent, portland cement) is employed as an additive mainly to control moisture damage in asphalt pavements. It is frequently required with certain siliceous aggregates which exhibit poor adhesion to asphalt, with the result that the asphalt-aggregate bond can be easily broken by water. However, the routine use of hydrated lime is unnecessary unless field experience or laboratory testing indicate that its addition will improve the resistance of the pavement to moisture damage.

Hydrated lime may be added to the aggregate at the hot-mix facility as a dry powder, or the aggregate may be pretreated with a slurry of hydrated lime in water. The rate of addition is usually one or two percent by weight of the aggregate.

Appendix A Miscellaneous Tables

Table A-1 Conversion Factors (Customary to metric units)

To convert from	To	Multiply by
acre	metre^2 (m^2)	4 046.856
acre	hectometre^2 (hm^2)	0.404 686
Atmosphere (technical = 1kgf/cm^2)	kilopascal (kPa)	98.066 50
barrel (42 gal.)	decimetre^3 (dm^3) or litre (l)	158.987 3
BTU (International Table)	kilojoule (kJ)	1.055 056
bushel	decimetre^3 (dm^3)	35.239 1
dyne	micronewton (μN)	10.000 0
dyne/centimetre^2	pascal (Pa)	0.100 0
Fahrenheit (temperature)	Celsius ($^\circ$C)	$t_c = (t_f-32)/1.8$
foot	metre (m)	0.304 80
foot^2	metre^2 (m^2)	0.092 903
foot^3	metre^3 (m^3)	0.028 317
	litre (l)	28.317 0
foot-pound-force	joule (J)	1.355 818
foot/minute	metre/second (m/s)	0.005 08
foot/second^2	metre/second^2 (m/s^2)	0.304 80
gallon (U.S. liquid)	decimetre^3 (dm^3) or litre (l)	3.785 412
	metre^3 (m^3)	0.003 785
gallon/minute	decimetre^3/second (dm^3/s) or litre/second (l/s)	0.063 09
gallon/yard^2	decimetre^3/metre^2 (dm^3/m^2) or litre/metre^2 (l/m^2)	4.527 314
horsepower (electric)	kilowatt (kW)	0.746 0
inch	millimetre (mm)	25.400 0
inch^2	centimetre^2 (cm^2)	6.451 60
inch^2	millimetre^2 (mm^2)	645.160 0
inch^3	centimetre^3 (cm^3)	16.387 06
inch/second	metre/second (m/s)	0.025 40
inch of mercury (60°F)	pascal (Pa)	3 376.85
inch/second^2	metre/second^2 (m/s^2)	0.025 40
kilogram (kg)	ton (metric)	0.001 00
kip (1 000 lbf)	kilonewton (kN)	4.448 222
kip/inch^2	megapascal (MPa)	6.894 757
mile (U.S. statute)	kilometre (km)	1.609 344
mile^2	kilometre^2 (km^2)	2.589 988
mile/hour	kilometre/hour (km/hr)	1.609 344
minute (angle)	radian (rad)	0.000 290 89
ounce-force	newton (N)	0.278 013 9
ounce-mass	gram (g)	28.349 52
ounce-fluid	centimetre^3 (cm^3)	29.573 53
	litre (l)	0.029 574
pint (U.S. liquid)	litre (l)	0.473 176 5
poise (absolute viscosity)	pascal-second (Pa-s)	0.100 000
pound-force (lbf)	newton (N)	4.448 222
	kilonewton (kN)	0.004 448
pound-force-inch	newton-metre (N.m)	0.112 984 8
pound-force/foot^2	pascal (Pa)	47.880 26
pound-force/inch^2 (psi)	kilopascal (kPa)	6.894 757
pound-mass	kilogram (kg)	0.453 592 4
pound-mass/foot^2	kilogram/metre^2 (kg/m^2)	4.882 428

(Continued)

Table A-1 Conversion Factors (Continued)

To convert from	To	Multiply by
pound-mass/foot^3	kilogram/metre^3 (kg/m^3)	16.018 46
	megagram/metre^3 (Mg/m^3)	0.016 018
pound-mass/inch^3	kilogram/decimetre^3 (kg/dm^3)	27.679 90
pound-mass/gallon (U.S. liquid)	kilogram/metre^3 (kg/m^3)	119.826 4
	kilogram/decimetre^3 (kg/dm^3)	0.119 826
psi	kilopascal (kPa)	6.894 757
quart (U.S. liquid)	decimetre^3 (dm^3) or litre (l)	0.946 352 9
ton (metric)	kilogram (kg)	1 000.000 0
ton (short-2 000 lb)	kilogram (kg)	907.184 7
ton (long-2 240 lb)	kilogram (kg)	1 016.046 1
ton-mass/yard^3	kilogram/metre^3 (kg/m^3)	1 186.552 7
yard	metre (m)	0.914 40
yard^2	metre^2 (m^2)	0.836 127 4
yard^3	metre^3 (m^3)	0.764 554 9

Table A-2 Area in Square Metres of Road Surface for Various Road Widths

Road Width	Per Lineal Metre	Per 50 m	Per Kilometre
2 m	2 m^2	100 m^2	2 000 m^2
2.5	2.5	125	2 500
3	3	150	3 000
3.5	3.5	175	3 500
4	4	200	4 000
4.5	4.5	225	4 500
5	5	250	5 000
5.5	5.5	275	5 500
6	6	300	6 000
6.5	6.5	325	6 500
7	7	350	7 000
7.5	7.5	375	7 500
8	8	400	8 000
8.5	8.5	425	8 500
9	9	450	9 000
9.5	9.5	475	9 500
10	10	500	10 000
10.5	10.5	525	10 500
11	11	550	11 000
11.5	11.5	575	11 500
12	12	600	12 000
15	15	750	15 000
20	20	1 000	20 000
25	25	1 250	25 000

Table A-3 Area in Square Yards of Road Surface for Various Road Widths

Road Width	Per Lineal Foot	Per 100 ft	Per Mile
6 ft	0.67	66.67	3,520
7	0.78	77.78	4,107
8	0.89	88.89	4,693
9	1.00	100.00	5,280
10	1.11	111.11	5,867
11	1.22	122.22	6,453
12	1.33	133.33	7,040
13	1.44	144.44	7,627
14	1.56	155.56	8,213
15	1.67	166.67	8,800
16	1.78	177.78	9,387
17	1.89	188.89	9,973
18	2.00	200.00	10,560
20	2.22	222.22	11,733
22	2.44	244.44	12,907
24	2.67	266.67	14,080
25	2.78	277.78	14,667
26	2.89	288.89	15,253
28	3.11	311.11	16,427
30	3.33	333.33	17,600
32	3.56	355.56	18,773
34	3.78	377.78	19,947
36	4.00	400.00	21,120
38	4.22	422.22	22,293
40	4.44	444.44	23,467
50	5.56	555.56	29,333
60	6.67	666.67	35,200
70	7.78	777.78	41,067
75	8.33	833.33	44,000
80	8.89	888.89	46,933

Table A-4 Mass and Volume Relations for Various Types of Compacted Asphalt Pavements

	kg/dm^3	kg/m^3	kg/m^2/cm depth	
Note: Because of the considerable variations of specific gravity, gradation, and other characteristics of mineral aggregates, mass per unit volume of compacted asphalt pavement varies considerably. Exact mass per unit volume should be determined in the laboratory from samples taken from the pavement or prepared in the laboratory with the same materials as used in the field.	1.60	1 600	16	
	1.70	1 700	17	
	1.80	1 800	18	
	1.90	1 900	19	
	2.00	2 000	20	
	2.10	2 100	21	
	2.20	2 200	22	
	2.30	2 300	23	
	2.40	2 400	24	
	2.50	2 500	25	
	2.60	2 600	26	
	Range	Range	Range	Frequently Used for Preliminary Estimate
Penetration Macadam	1.75-2.15	1 750-2 150	17.5-21.5	20.0
Open Graded	1.85-2.25	1 850-2 250	18.5-22.5	21.0
Coarse Graded	2.10-2.40	2 100-2 400	21.0-24.0	22.5
Dense Graded	2.15-2.50	2 150-2 500	21.5-25.0	23.5
Fine Graded	2.10-2.40	2 100-2 400	21.0-24.0	22.5
Stone Sheet	2.10-2.40	2 100-2 400	21.0-24.0	22.5
Sand Sheet	1.90-2.25	1 900-2 250	19.0-22.5	21.0
Fine Sheet	1.90-2.25	1 900-2 250	19.0-22.5	21.0
Mixed-in-place Macadam	1.75-2.15	1 750-2 150	17.5-21.5	20.0
Mixed-in-place Dense Graded	1.75-2.15	1 750-2 150	17.5-21.5	20.0
Mixed-in-place Sand Asphalt	1.60-2.00	1 600-2 000	16.0-20.0	18.0

Table A-5 Weight and Volume Relations for Various Types of Compacted Asphalt Pavements

	lb/ft^3	lb/yd^3	lb/yd^2/in. depth	
Note: Because of the considerable variations of specific gravity, gradation, and other characteristics of mineral aggregates, weight per unit volume of compacted asphalt pavement varies considerably. Exact weights per unit volume should be determined in the laboratory from samples taken from the pavement or prepared in the laboratory with the same materials as used in the field.	100	2700	75	
	105	2835	79	
	110	2970	82	
	115	3105	86	
	120	3240	90	
	125	3375	94	
	130	3510	97	
	135	3645	101	
	140	3780	105	
	145	3915	109	
	150	4050	112	
	155	4185	116	
	160	4320	120	
	Range	Range	Range	Frequently Used for Preliminary Estimate
Penetration Macadam	110-135	2970-3645	82-101	95
Open Graded	115-140	3105-3780	86-105	100
Coarse Graded	130-150	3510-4050	97-112	105
Dense Graded	135-155	3645-4185	101-116	110
Fine Graded	130-150	3510-4050	97-112	105
Stone Sheet	130-150	3510-4050	97-112	105
Sand Sheet	120-140	3240-3780	90-105	100
Fine Sheet	120-140	3240-3780	90-105	100
Mixed-in-place Macadam	110-135	2970-3645	82-101	95
Mixed-in-place Dense Graded	110-135	2970-3645	82-101	95
Mixed-in-place Sand Asphalt	100-125	2700-3375	75-94	85

Table A-6 Kilograms Per Square Metre of Material Required for Various Depths and Masses of Materials in Kilograms Per Cubic Metre

kg/m^3	Depth - Centimetres											
	2	4	6	8	10	12	14	16	18	20	22	24
1 100	22	44	66	88	110	132	154	176	198	220	242	264
1 200	24	48	72	96	120	144	168	192	216	240	264	288
1 300	26	52	78	104	130	156	182	208	234	260	286	312
1 400	28	56	84	112	140	168	196	224	252	280	308	336
1 500	30	60	90	120	150	180	210	240	270	300	330	360
2 000	40	80	120	160	200	240	280	320	360	400	440	480
2 500	50	100	150	200	250	300	350	400	450	500	550	600
3 000	60	120	180	240	300	360	420	480	540	600	660	720

NOTE: Formula used for calculations: $q = \frac{DW}{100}$

where: q = Quantity of material in kg/m^2
D = Depth in cm
W = Mass of material in kg/m^3

Table A-7 Pounds Per Square Yard of Material Required for Various Depths and Weights of Materials in Pounds Per Cubic Yard

lb/yd³	Depth - Inches											
	1	2	3	4	5	6	7	8	9	10	11	12
2,000	55.5	111.1	166.6	222.2	277.7	333.2	388.8	444.3	499.9	555.4	610.9	666.5
2,500	69.4	138.9	208.3	277.7	347.1	416.6	486.0	555.4	624.8	694.3	763.7	833.1
3,000	83.3	166.6	250.0	333.2	416.6	500.0	583.2	666.5	749.8	833.1	916.4	999.7
3,500	97.2	194.4	291.6	388.8	486.0	583.2	680.4	777.6	874.8	972.0	1069.1	1166.3
4,000	111.1	222.2	333.2	444.3	555.4	666.5	777.6	888.6	999.7	1110.8	1221.9	1333.0
4,500	125.0	249.9	374.9	499.9	624.8	749.8	874.8	999.7	1124.7	1249.7	1374.6	1499.6
5,000	138.9	277.7	416.6	555.4	694.3	833.1	972.0	1110.8	1249.7	1388.5	1527.4	1666.2

NOTE: Formula used for calculations: $q = \frac{D}{36}W$

where: q = Quantity of material in lb/yd^2
D = Depth in inches
W = Weight of material in lb/yd^3

Table A-8 Tonnes of Material Required Per 50 Linear Metres for Various Widths and Kilograms Per Square Metre

kg/m^2	Width - Metres											
	1	2	3	4	5	6	7	8	9	10	15	20
5	0.25	0.50	0.75	1.00	1.25	1.50	1.75	2.00	2.25	2.50	3.75	5.00
10	0.50	1.00	1.50	2.00	2.50	3.00	3.50	4.00	4.50	5.00	7.50	10.00
20	1.00	2.00	3.00	4.00	5.00	6.00	7.00	8.00	9.00	10.00	15.00	20.00
30	1.50	3.00	4.50	6.00	7.50	9.00	10.50	12.00	13.50	15.00	22.50	30.00
40	2.00	4.00	6.00	8.00	10.00	12.00	14.00	16.00	18.00	20.00	30.00	40.00
50	2.50	5.00	7.50	10.00	12.50	15.00	17.50	20.00	22.50	25.00	37.50	50.00
60	3.00	6.00	9.00	12.00	15.00	18.00	21.00	24.00	27.00	30.00	45.00	60.00
70	3.50	7.00	10.50	14.00	17.50	21.00	24.50	28.00	31.50	35.00	52.50	70.00
80	4.00	8.00	12.00	16.00	20.00	24.00	28.00	32.00	36.00	40.00	60.00	80.00
90	4.50	9.00	13.50	18.00	22.50	27.00	31.50	36.00	40.50	45.00	67.50	90.00
100	5.00	10.00	15.00	20.00	25.00	30.00	35.00	40.00	45.00	50.00	75.00	100.00
200	10.00	20.00	30.00	40.00	50.00	60.00	70.00	80.00	90.00	100.00	150.00	200.00
300	15.00	30.00	45.00	60.00	75.00	90.00	105.00	120.00	135.00	150.00	225.00	300.00
400	20.00	40.00	60.00	80.00	100.00	120.00	140.00	160.00	180.00	200.00	300.00	400.00
500	25.00	50.00	75.00	100.00	125.00	150.00	175.00	200.00	225.00	250.00	375.00	500.00

NOTE: Formula used for calculation: $M = \frac{RWL}{1\ 000} = 0.001\ RWL$

where: M = Mass of material in tonnes per 50 metres
R = Rate of application, kg/m^2
W = Width of application, metres
L = Length of section, 50 metres

Table A-9 Tons of Material Required Per 100 Linear Feet for Various Widths and Pounds Per Square Yard

lb/yd^2	Width - Feet														
	1	2	3	4	5	6	7	8	9	10	20	30	40	50	60
10	0.06	0.11	0.17	0.22	0.28	0.33	0.39	0.44	0.50	0.56	1.11	1.67	2.22	2.78	3.33
20	0.11	0.22	0.33	0.44	0.56	0.67	0.78	0.89	1.00	1.11	2.22	3.33	4.44	5.56	6.67
30	0.17	0.33	0.50	0.67	0.83	1.00	1.17	1.33	1.50	1.67	3.33	5.00	6.67	8.33	10.00
40	0.22	0.44	0.67	0.89	1.11	1.33	1.56	1.78	2.00	2.22	4.44	6.67	8.89	11.11	13.33
50	0.28	0.56	0.83	1.11	1.39	1.67	1.94	2.22	2.50	2.78	5.56	8.33	11.11	13.89	16.67
60	0.33	0.67	1.00	1.33	1.67	2.00	2.33	2.67	3.00	3.33	6.67	10.00	13.33	16.67	20.00
70	0.39	0.78	1.17	1.56	1.94	2.33	2.72	3.11	3.50	3.89	7.78	11.67	15.56	19.44	23.33
80	0.44	0.89	1.33	1.78	2.22	2.67	3.11	5.56	4.00	4.44	8.89	13.33	17.78	22.22	26.67
90	0.50	1.00	1.50	2.00	2.50	3.00	3.50	4.00	4.50	5.00	10.00	15.00	20.00	25.00	30.00
100	0.56	1.11	1.67	2.22	2.78	3.33	3.89	4.44	5.00	5.56	11.11	16.67	22.22	27.78	33.33
200	1.11	2.22	3.33	4.44	5.56	6.67	7.78	8.89	10.00	11.11	22.22	33.33	44.44	55.56	66.67
300	1.67	3.33	5.00	6.67	8.33	10.00	11.67	13.33	15.00	16.67	33.33	50.00	66.67	83.33	100.00
400	2.22	4.44	6.67	8.89	11.11	13.33	15.56	17.78	20.00	22.22	44.44	66.67	88.89	111.11	133.33
500	2.78	5.56	8.33	11.11	13.89	16.67	19.44	22.22	25.00	27.78	55.56	83.33	111.11	138.89	166.67
600	3.33	6.67	10.00	13.33	16.67	20.00	23.33	26.67	30.00	33.33	66.67	100.00	133.33	166.67	200.00
700	3.89	7.78	11.67	15.56	19.44	23.33	27.22	31.11	35.00	38.89	77.78	116.69	155.56	194.44	233.33
800	4.44	8.89	13.33	17.78	22.22	26.67	31.11	35.56	40.00	44.44	88.89	133.33	177.78	222.22	266.67
900	5.00	10.00	15.00	20.00	25.00	30.00	35.00	40.00	45.00	50.00	100.00	150.00	200.00	250.00	300.00
1,000	5.56	11.11	16.67	22.22	27.78	33.33	38.89	44.44	50.00	55.56	111.11	166.67	222.22	277.78	333.33

NOTE: Formula used for calculation: $w = \left(\frac{W}{3}\right)\left(\frac{100}{3}\right)\left(\frac{R}{2000}\right) = 0.005556\,RW$

where: w = Weight of material in tons per 100 feet
R = Rate of application, lb/yd^2
W = Width of application, feet

Table A-10 Tonnes of Material Required Per Kilometre for Various Widths and Kilograms Per Square Metre

kg/m²	Width - Metres											
	1	2	3	4	5	6	7	8	9	10	15	20
5	5.0	10.0	15.0	20.0	25.0	30.0	35.0	40.0	45.0	50.0	75.0	100.0
10	10.0	20.0	30.0	40.0	50.0	60.0	70.0	80.0	90.0	100.0	150.0	200.0
20	20.0	40.0	60.0	80.0	100.0	120.0	140.0	160.0	180.0	200.0	300.0	400.0
30	30.0	60.0	90.0	120.0	150.0	180.0	210.0	240.0	270.0	300.0	450.0	600.0
40	40.0	80.0	120.0	160.0	200.0	240.0	280.0	320.0	360.0	400.0	600.0	800.0
50	50.0	100.0	150.0	200.0	250.0	300.0	350.0	400.0	450.0	500.0	750.0	1 000.0
60	60.0	120.0	180.0	240.0	300.0	360.0	420.0	480.0	540.0	600.0	900.0	1 200.0
70	70.0	140.0	210.0	280.0	350.0	420.0	490.0	560.0	630.0	700.0	1 050.0	1 400.0
80	80.0	160.0	240.0	320.0	400.0	480.0	560.0	640.0	720.0	800.0	1 200.0	1 600.0
90	90.0	180.0	270.0	360.0	450.0	540.0	630.0	720.0	810.0	900.0	1 350.0	1 800.0
100	100.0	200.0	300.0	400.0	500.0	600.0	700.0	800.0	900.0	1 000.0	1 500.0	2 000.0
200	200.0	400.0	600.0	800.0	1 000.0	1 200.0	1 400.0	1 600.0	1 800.0	2 000.0	3 000.0	4 000.0
300	300.0	600.0	900.0	1 200.0	1 500.0	1 800.0	2 100.0	2 400.0	2 700.0	3 000.0	4 500.0	6 000.0
400	400.0	800.0	1 200.0	1 600.0	2 000.0	2 400.0	2 800.0	3 200.0	3 600.0	4 000.0	6 000.0	8 000.0
500	500.0	1 000.0	1 500.0	2 000.0	2 500.0	3 000.0	3 500.0	4 000.0	4 500.0	5 000.0	7 500.0	10 000.0

NOTE: Formula used for calculation: $M = RWL$

where:
M = Mass of material, tonnes per kilometre
R = Rate of application, kg/m^2
W = Width of application, metres
L = Length of section, 1 kilometre

Table A-11 Tons of Material Required Per Mile for Various Widths and Pounds Per Square Yard

lb/yd^2	Width - Feet													
	1	2	3	4	5	6	7	8	9	10	20	30	40	50	60
10	2.9	5.9	8.8	11.7	14.7	17.6	20.5	23.5	26.4	29.3	58.7	88.0	117.3	146.7	176.0
20	5.9	11.7	17.6	23.5	29.3	35.2	41.1	46.9	52.8	58.7	117.3	176.0	234.7	293.3	352.0
30	8.8	17.6	26.4	35.2	44.0	52.8	61.6	70.4	79.2	88.0	176.0	264.0	352.0	440.0	527.9
40	11.7	23.5	35.2	46.9	58.7	70.4	82.1	93.9	105.6	117.3	234.7	352.0	469.3	586.7	704.0
50	14.7	29.3	44.0	58.7	73.3	88.0	102.7	117.3	132.0	146.7	293.3	440.0	586.7	733.3	880.0
60	17.6	35.2	52.8	70.4	88.0	105.6	123.2	140.8	158.4	176.0	352.0	528.0	704.0	880.0	1,056.0
70	20.5	41.1	61.6	82.1	102.7	123.2	143.7	164.3	184.8	205.3	410.7	616.0	821.3	1,026.7	1,232.0
80	23.5	46.9	70.4	93.9	117.3	140.8	164.3	187.7	211.2	234.7	469.3	704.0	938.7	1,173.3	1,408.0
90	26.4	52.8	79.2	105.6	132.0	158.4	184.8	211.2	237.6	264.0	528.0	792.0	1,056.0	1,320.0	1,584.0
100	29.3	58.7	88.0	117.3	146.7	176.0	205.3	234.7	264.0	293.3	586.7	880.0	1,173.3	1,466.7	1,760.0
200	58.7	117.3	176.0	234.7	293.3	352.0	410.7	469.3	528.0	586.7	1,173.3	1,760.0	2,346.7	2,933.3	3,520.0
300	88.0	176.0	264.0	352.0	440.0	528.0	616.0	704.0	792.0	880.0	1,760.0	2,640.0	3,520.0	4,400.0	5,280.0
400	117.3	234.7	352.0	469.3	586.7	704.0	821.3	938.7	1,056.0	1,173.3	2,346.7	3,520.0	4,693.3	5,866.7	7,040.0
500	146.7	293.3	440.0	586.7	733.3	880.0	1,026.7	1,173.3	1,320.0	1,466.7	2,933.3	4,400.0	5,866.7	7,333.3	8,800.0
600	176.0	352.0	528.0	704.0	880.0	1,056.0	1,232.0	1,408.0	1,584.0	1,760.0	3,520.0	5,280.0	7,040.0	8,800.0	10,560.0
700	205.3	410.7	616.0	821.3	1,026.7	1,232.0	1,437.3	1,642.7	1,848.0	2,053.3	4,106.7	6,160.0	8,213.3	10,266.7	12,320.0
800	234.7	469.3	704.0	938.7	1,173.3	1,408.0	1,642.7	1,877.3	2,112.0	2,346.7	4,693.3	7,040.0	9,386.7	11,733.3	14,080.0
900	264.0	528.0	792.0	1,056.0	1,320.0	1,584.0	1,848.0	2,112.0	2,376.0	2,640.0	5,280.0	7,920.0	10,560.0	13,200.0	15,840.0
1,000	293.3	586.7	880.0	1,173.3	1,466.7	1,760.0	2,053.3	2,346.7	2,640.0	2,933.3	5,866.7	8,800.0	11,733.3	14,666.7	17,600.0

NOTE: Formula used for calculation: $w = \left(\frac{W}{3}\right)\left(\frac{5280}{3}\right)\left(\frac{R}{2000}\right) = 0.2933\,RW$

where: w = Weight of material, tons per mile
R = Rate of application, lb/yd^2
W = Width of application, feet

Table A-12 Linear Metres Covered by One Tonne of Material for Various Widths and Kilograms Per Square Metre

kg/m²	Width - Metres											
	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7	7.5
5	100.0	80.0	66.7	57.1	50.0	44.4	40.0	36.4	33.3	30.8	28.6	26.7
10	50.0	40.0	33.3	28.6	25.0	22.2	20.0	18.2	16.7	15.4	14.3	13.3
15	33.3	26.7	22.2	19.0	16.7	14.8	13.3	12.1	11.1	10.3	9.5	8.9
20	25.0	20.0	16.7	14.3	12.5	11.1	10.0	9.1	8.3	7.7	7.1	6.7
25	20.0	16.0	13.3	11.4	10.0	8.9	8.0	7.3	6.7	6.2	5.7	5.3
30	16.7	13.3	11.1	9.5	8.3	7.4	6.7	6.1	5.6	5.1	4.8	4.4
35	14.3	11.4	9.5	8.2	7.1	6.3	5.7	5.2	4.8	4.4	4.1	3.8
40	12.5	10.0	8.3	7.1	6.3	5.6	5.0	4.5	4.2	3.8	3.6	3.3
45	11.1	8.9	7.4	6.3	5.6	4.9	4.4	4.0	3.7	3.4	3.2	3.0
50	10.0	8.0	6.7	5.7	5.0	4.4	4.0	3.6	3.3	3.1	2.9	2.7
60	8.3	6.7	5.6	4.8	4.2	3.7	3.3	3.0	2.8	2.6	2.4	2.2
70	7.1	5.7	4.8	4.1	3.6	3.2	2.9	2.6	2.4	2.2	2.0	1.9
80	6.3	5.0	4.2	3.6	3.1	2.8	2.5	2.3	2.1	1.9	1.8	1.7
90	5.6	4.4	3.7	3.2	2.8	2.5	2.2	2.0	1.9	1.7	1.6	1.5
100	5.0	4.0	3.3	2.9	2.5	2.2	2.0	1.8	1.7	1.5	1.4	1.3
200	2.5	2.0	1.7	1.4	1.3	1.1	1.0	0.9	0.8	0.8	0.7	0.7
300	1.7	1.3	1.1	1.0	0.8	0.7	0.7	0.6	0.6	0.5	0.5	0.4
400	1.3	1.0	0.8	0.7	0.6	0.6	0.5	0.5	0.4	0.4	0.4	0.3
500	1.0	0.8	0.7	0.6	0.5	0.4	0.4	0.4	0.3	0.3	0.3	0.3
600	0.8	0.7	0.6	0.5	0.4	0.4	0.3	0.3	0.3	0.3	0.2	0.2

NOTE: Formula used for calculation: $L = \frac{1\ 000}{RW}$

where: L = Linear metres covered by one tonne of material
R = Rate of spread, kg/m²
W = Width of spread, metres

Table A-13 Linear Feet Covered by One Ton of Material for Various Widths and Pounds Per Square Yard

lb/yd²	Width - Feet									
	8	9	10	11	12	13	14	15	16	17
20	112.5	100.0	90.0	81.8	75.0	69.2	64.3	60.0	56.3	52.9
25	90.0	80.0	72.0	65.5	60.0	55.4	51.4	48.0	45.0	42.4
30	75.0	66.7	60.0	54.5	50.0	46.2	42.9	40.0	37.5	35.3
35	64.3	57.1	51.4	46.8	42.9	39.6	36.7	34.3	32.1	30.3
40	56.3	50.0	45.0	40.9	37.5	34.6	32.1	30.0	28.1	26.5
45	50.0	44.4	40.0	36.4	33.3	30.8	28.6	26.7	25.0	23.5
50	45.0	40.0	36.0	32.7	30.0	27.7	25.7	24.0	22.5	21.2
60	37.5	33.3	30.0	27.3	25.0	23.1	21.4	20.0	18.8	17.6
70	32.1	28.6	25.7	23.4	21.4	19.8	18.4	17.1	16.1	15.1
80	28.1	25.0	22.5	20.5	18.8	17.3	16.1	15.0	14.1	13.2
90	25.0	22.2	20.0	18.2	16.7	15.4	14.3	13.3	12.5	11.8
100	22.5	20.0	18.0	16.4	15.0	13.8	12.9	12.0	11.3	10.6
150	15.0	13.3	12.0	10.9	10.0	9.2	8.6	8.0	7.5	7.1
200	11.3	10.0	9.0	8.2	7.5	6.9	6.4	6.0	5.6	5.3
250	9.0	8.0	7.2	6.5	6.0	5.5	5.1	4.8	4.5	4.2
300	7.5	6.7	6.0	5.5	5.0	4.6	4.3	4.0	3.8	3.5
400	5.6	5.0	4.5	4.1	3.8	3.5	3.2	3.0	2.8	2.6
500	4.5	4.0	3.6	3.3	3.0	2.8	2.6	2.4	2.2	2.1
600	3.7	3.3	3.0	2.7	2.5	2.3	2.1	2.0	1.9	1.8
700	3.2	2.9	2.6	2.3	2.1	2.0	2.0	1.7	1.6	1.5
800	2.8	2.5	2.3	2.0	1.9	1.7	1.6	1.5	1.4	1.3
900	2.5	2.2	2.0	1.8	1.7	1.5	1.4	1.3	1.2	1.2
1,000	2.3	2.0	1.8	1.6	1.5	1.4	1.3	1.2	1.1	1.1
1,100	2.0	1.8	1.6	1.5	1.4	1.3	1.2	1.1	1.0	1.0

NOTE: Formula used for calculation: $L = \frac{2{,}000(9)}{RW} = \frac{18{,}000}{RW}$

where: L = Linear feet covered by one ton of material
R = Rate of spread, lb/ft²
W = Width of spread, feet

Table A-14 Cubic Metres of Material Required for Various Widths and Depths Per 50 Linear Metres and Per Kilometre

	Width, Metres	Depth - Centimetres											
		2	4	6	8	10	12	14	16	18	20	22	24
PER 50 LINEAR METRES	1	1.00	2.00	3.00	4.00	5.00	6.00	7.00	8.00	9.00	10.00	11.00	12.00
	2	2.00	4.00	6.00	8.00	10.00	12.00	14.00	16.00	18.00	20.00	22.00	24.00
	3	3.00	6.00	9.00	12.00	15.00	18.00	21.00	24.00	27.00	30.00	33.00	36.00
	4	4.00	8.00	12.00	16.00	20.00	24.00	28.00	32.00	36.00	40.00	44.00	48.00
	5	5.00	10.00	15.00	20.00	25.00	30.00	35.00	40.00	45.00	50.00	55.00	60.00
	6	6.00	12.00	18.00	24.00	30.00	36.00	42.00	48.00	54.00	60.00	66.00	72.00
	7	7.00	14.00	21.00	28.00	35.00	42.00	49.00	56.00	63.00	70.00	77.00	84.00
	8	8.00	16.00	24.00	32.00	40.00	48.00	56.00	64.00	72.00	80.00	88.00	96.00
	9	9.00	18.00	27.00	36.00	45.00	54.00	63.00	72.00	81.00	90.00	99.00	108.00
	10	10.00	20.00	30.00	40.00	50.00	60.00	70.00	80.00	90.00	100.00	110.00	120.00
	15	15.00	30.00	45.00	60.00	75.00	90.00	105.00	120.00	135.00	150.00	165.00	180.00
	20	20.00	40.00	60.00	80.00	100.00	120.00	140.00	160.00	180.00	200.00	220.00	240.00
	25	25.00	50.00	75.00	100.00	125.00	150.00	175.00	200.00	225.00	250.00	275.00	300.00
	30	30.00	60.00	90.00	120.00	150.00	180.00	210.00	240.00	270.00	300.00	330.00	360.00
PER KILOMETRE	1	20.00	40.00	60.00	80.00	100.00	120.00	140.00	160.00	180.00	200.00	220.00	240.00
	2	40.00	80.00	120.00	160.00	200.00	240.00	280.00	320.00	360.00	400.00	440.00	480.00
	3	60.00	120.00	180.00	240.00	300.00	360.00	420.00	480.00	540.00	600.00	660.00	720.00
	4	80.00	160.00	240.00	320.00	400.00	480.00	560.00	640.00	720.00	800.00	880.00	760.00
	5	100.00	200.00	300.00	400.00	500.00	600.00	700.00	800.00	900.00	1 000.00	1 100.00	1 200.00
	6	120.00	240.00	360.00	480.00	600.00	720.00	840.00	960.00	1 080.00	1 200.00	1 320.00	1 440.00
	7	140.00	280.00	420.00	560.00	700.00	840.00	980.00	1 120.00	1 260.00	1 400.00	1 540.00	1 680.00
	8	160.00	320.00	480.00	640.00	800.00	960.00	1 120.00	1 280.00	1 440.00	1 600.00	1 760.00	1 920.00
	9	180.00	360.00	540.00	720.00	900.00	1 080.00	1 260.00	1 440.00	1 620.00	1 800.00	1 980.00	2 160.00
	10	200.00	400.00	600.00	800.00	1 000.00	1 200.00	1 400.00	1 600.00	1 800.00	2 000.00	2 200.00	2 400.00
	15	300.00	600.00	900.00	1 200.00	1 500.00	1 800.00	2 100.00	2 400.00	2 700.00	3 000.00	3 300.00	3 600.00
	20	400.00	800.00	1 200.00	1 600.00	2 000.00	2 400.00	2 800.00	3 200.00	3 600.00	4 000.00	4 400.00	4 800.00
	25	500.00	1 000.00	1 500.00	2 000.00	2 500.00	3 000.00	3 500.00	4 000.00	4 500.00	5 000.00	5 500.00	6 000.00
	30	600.00	1 200.00	1 800.00	2 400.00	3 000.00	3 600.00	4 200.00	4 800.00	5 400.00	6 000.00	6 600.00	7 200.00

NOTE: Formulas used for calculations: Per 50 linear metres: $q = \frac{DW}{100}(50) = 0.50\,DW$

Per kilometre: $q = \frac{DW}{100}(1\ 000) = 10\ DW$

where: q = Quantity of material, cubic metres
D = Depth, centimetres
W = Width, metres

Table A-15 Cubic Yards of Material Required for Various Widths and Depths Per 100 Linear Feet and Per Mile

	Width, Feet	Depth - Inches											
		1	2	3	4	5	6	7	8	9	10	11	12
PER 100 LINEAR FEET	1	0.31	0.62	0.93	1.23	1.54	1.85	2.16	2.47	2.78	3.09	3.40	3.70
	2	0.62	1.23	1.85	2.47	3.09	3.70	4.32	4.94	5.56	6.17	6.79	7.41
	3	0.93	1.85	2.78	3.70	4.63	5.56	6.48	7.41	8.33	9.26	10.20	11.10
	4	1.23	2.47	3.70	4.94	6.17	7.41	8.64	9.88	11.10	12.30	13.60	14.80
	5	1.54	3.09	4.63	6.17	7.72	9.26	10.80	12.30	13.90	15.40	17.00	18.50
	6	1.85	3.70	5.56	7.41	9.26	11.10	13.00	14.80	16.70	18.50	20.40	22.20
	7	2.16	4.32	6.48	8.64	10.80	13.00	15.10	17.30	19.40	21.60	23.80	25.90
	8	2.47	4.94	7.41	9.88	12.30	14.80	17.30	19.80	22.20	24.70	27.20	29.60
	9	2.78	5.56	8.33	11.10	13.90	16.70	19.40	22.20	25.00	27.80	30.60	33.30
	10	3.09	6.17	9.26	12.30	15.40	18.50	21.60	24.70	27.80	30.90	34.00	37.00
	20	6.17	12.30	18.50	24.70	30.90	37.00	43.20	49.40	55.60	61.70	67.90	74.10
	30	9.26	18.50	27.80	37.00	46.30	55.60	64.80	74.10	83.30	92.60	102.00	111.00
	40	12.30	24.70	37.00	49.40	61.70	74.10	86.40	98.80	111.00	123.00	136.00	148.00
	50	15.40	30.90	46.30	61.70	77.20	92.60	108.00	123.00	139.00	154.00	170.00	185.00
	60	18.50	37.00	55.60	74.10	92.60	111.00	130.00	148.00	167.00	185.00	204.00	222.00
	70	21.60	43.20	64.80	86.40	108.00	130.00	151.00	173.00	194.00	216.00	238.00	259.00
	80	24.70	49.40	74.10	98.80	123.00	148.00	173.00	198.00	222.00	247.00	272.00	296.00
	90	27.80	55.60	83.30	111.00	139.00	167.00	194.00	222.00	250.00	278.00	306.00	333.00
	100	30.90	61.70	92.60	123.00	154.00	185.00	216.00	247.00	278.00	309.00	340.00	370.00
PER MILE	1	16.30	32.60	48.90	65.20	81.50	97.80	114.00	130.00	147.00	163.00	179.00	196.00
	2	32.60	65.20	97.80	130.00	163.00	196.00	228.00	261.00	293.00	326.00	359.00	391.00
	3	48.90	97.80	147.00	196.00	244.00	293.00	342.00	391.00	440.00	489.00	538.00	587.00
	4	65.20	130.00	196.00	261.00	326.00	391.00	456.00	521.00	587.00	652.00	717.00	782.00
	5	81.50	163.00	244.00	326.00	407.00	489.00	570.00	652.00	733.00	815.00	896.00	978.00
	6	97.80	196.00	293.00	391.00	489.00	587.00	684.00	782.00	880.00	978.00	1,076.00	1,173.00
	7	114.00	228.00	342.00	456.00	570.00	684.00	799.00	913.00	1,027.00	1,141.00	1,255.00	1,369.00
	8	130.00	261.00	391.00	521.00	652.00	782.00	913.00	1,043.00	1,173.00	1,304.00	1,434.00	1,564.00
	9	147.00	293.00	440.00	587.00	733.00	880.00	1,027.00	1,173.00	1,320.00	1,467.00	1,613.00	1,760.00
	10	163.00	326.00	489.00	652.00	815.00	978.00	1,141.00	1,304.00	1,467.00	1,630.00	1,793.00	1,956.00
	20	326.00	652.00	978.00	1,304.00	1,630.00	1,956.00	2,281.00	2,607.00	2,933.00	3,259.00	3,585.00	3,911.00
	30	489.00	978.00	1,467.00	1,956.00	2,444.00	2,933.00	3,422.00	3,911.00	4,440.00	4,889.00	5,378.00	5,867.00
	40	652.00	1,304.00	1,956.00	2,607.00	3,259.00	3,911.00	4,563.00	5,215.00	5,867.00	6,519.00	7,170.00	7,822.00
	50	815.00	1,630.00	2,444.00	3,259.00	4,074.00	4,889.00	5,704.00	6,519.00	7,333.00	8,148.00	8,963.00	9,778.00
	60	978.00	1,956.00	2,933.00	3,911.00	4,889.00	5,867.00	6,844.00	7,822.00	8,800.00	9,778.00	10,756.00	11,733.00
	70	1,141.00	2,281.00	3,422.00	4,563.00	5,704.00	6,844.00	7,985.00	9,126.00	10,267.00	11,407.00	12,548.00	13,689.00
	80	1,304.00	2,607.00	3,911.00	5,215.00	6,519.00	7,822.00	9,126.00	10,430.00	11,733.00	13,037.00	14,341.00	15,644.00
	90	1,467.00	2,933.00	4,400.00	5,867.00	7,333.00	8,800.00	10,267.00	11,733.00	13,200.00	14,667.00	16,133.00	17,600.00
	100	1,630.00	3,259.00	4,889.00	6,519.00	8,148.00	9,778.00	11,407.00	13,037.00	14,667.00	16,296.00	17,926.00	19,556.00

NOTE: Formulas used for calculations:

Per 100 Lin Ft: $q = \left(\frac{D}{36}\right)\left(\frac{W}{3}\right)\left(\frac{100}{3}\right) = 0.3086\,DW$

Per Mile: $q = \left(\frac{D}{36}\right)\left(\frac{W}{3}\right)\left(\frac{5{,}280}{3}\right) = 16.2963\,DW$

where:
q = Quantity of material, cubic yards
D = Depth, inches
W = Width, feet
L = Length

Appendix B

Random Sampling Plans

B.01 SELECTING SAMPLING LOCATIONS IN TRUCKS HAULING ASPHALT MIXTURE (Procedure No. 1).—The following definitions apply (see also Figure B-1):

- *Lot*—a quantity of material that one desires to control. It may represent a day's production, a specified tonnage, a specified number of truckloads, a specified time period during production.
- *Sample*—a segment of a lot chosen to represent the total lot. It may represent any number of subsamples.
- *Subsample*—a segment of a sample, taken from a unit of the lot, i.e., a specified ton, a specified time, a specified truckload.
- *Sample Unit*—a portion of a subsample taken from a unit of a lot and combined with one or more other sample units to make up a subsample.

In this procedure the following steps are necessary to select the sampling locations:

(1) Select the lot size—it can be time (hours), an average day's production (tonnes), a selected tonnage (example: 2,000 tonnes), or a selected number of truckloads. (A lot size of a day's production is recommended for this procedure as being convenient and easy to randomize.)

(2) Select the number of samples desired per lot. One sample per lot, made up of *four* subsamples, is the minimum recommended.

(3) Select the number of locations in each truckload from which sampling units of asphalt mixtures will be taken to combine into one subsample. *Two* sampling units per subsample are recommended.

(4) Assign each truckload of mixture in the lot a number, beginning with 1 for the first truckload and number them successively to the highest number in the lot. Find the truckload numbers for sampling by the following procedure:

  - Place consecutively numbered (1 through   ) 25 mm (1 in.) square pieces of cardboard, equal to the number of truckloads in the lot, into a container (such as a bowl). Mix them thoroughly before each drawing.
  - Draw a number of cardboard squares from the container equal to the number of subsamples desired for the lot. The numerals on the cardboard squares will be the truckloads to be sampled.

(5) Choose for each subsample desired the location in the truckload for each of the sampling units. Use the following steps:

  - Divide the truck beds into equal quadrants and number then 1 through 4 in any order desired.

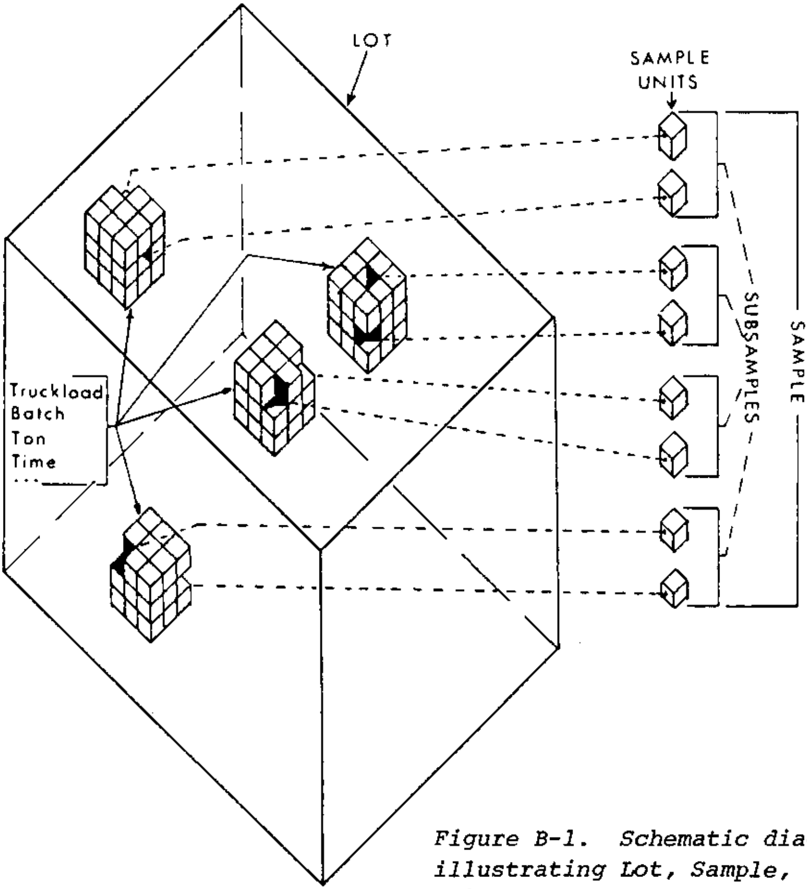

Figure B-1. Schematic diagram illustrating Lot, Sample, Subsample, and Sample Unit.

- Place four consecutively numbered (1 through 4) 25 mm (1 in.) square pieces of cardboard into a container (such as a bowl). Mix them thoroughly before each drawing.
- Draw out an amount of cardboard squares equal to the number of sample units desired. The numerals on each square drawn represent the quadrants from which the sample unit will be taken. Replace the cardboard squares and repeat this step for each sample unit of each subsample to be taken.

B.02 SELECTING SAMPLING LOCATIONS AT PAVEMENT SITE (Procedure No. 2).—Table B-1 contains random numbers for the general sampling procedure. To use this table for selecting locations for sampling or testing the following steps are necessary:

(1) *For compacted pavement sampling or testing locations,* use each day's run as a separate section.

(2) Determine the number of sampling locations within a section by selecting the maximum *average* longitudinal distance desired between samples and dividing the length of the section by the maximum average longitudinal distance.

(3) Select a column of random numbers in Table B-1 by placing 28 pieces of cardboard 25 mm (1 in.) square, numbered 1 through 28, into a container (such as a bowl), shaking them to get them thoroughly mixed, and drawing one out.

(4) Go to the column of random numbers identified with the number drawn from the container. In subcolumn A, locate all numbers equal to and less than the number of sampling locations per section desired.

(5) Multiply the total length of the section by the decimal values in subcolumn B, found opposite the numbers located in subcolumn A. Add the result to the station number at the beginning of the section to obtain the station of the sampling location.

(6) Multiply the total width of the proposed pavement in the section by the decimal values in subcolumn C, found opposite the numbers located in subcolumn A, then subtract one-half of the total width of the proposed pavement from the result to obtain the offest distance from the centerline to the sampling location. A positive (+) number will be the distance to the *right* of centerline and a negative (-) number will be the distance to the *left* of centerline. If only one lane of pavement is involved, the total width will be the lane width and the offset distance will be measured from the left edge of the lane.

Table B-1 Random Numbers for General Sampling Procedure

Col. No. 1			Col. No. 2			Col. No. 3			Col. No. 4			Col. No. 5			Col. No. 6			Col. No. 7		
A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C
15	.033	.576	05	.048	.879	21	.013	.220	18	.089	.716	17	.024	.863	30	.030	.901	12	.029	.386
21	.101	.300	17	.074	.156	30	.036	.853	10	.102	.330	24	.060	.032	21	.096	.198	18	.112	.284
23	.129	.916	18	.102	.191	10	.052	.746	14	.111	.925	26	.074	.639	10	.100	.161	20	.114	.848
30	.158	.434	06	.105	.257	25	.061	.954	28	.127	.840	07	.167	.512	29	.133	.388	03	.121	.656
24	.177	.397	28	.179	.447	29	.062	.507	24	.132	.271	28	.194	.776	24	.138	.062	13	.178	.640
11	.202	.271	26	.187	.844	18	.087	.887	19	.285	.899	03	.219	.166	20	.168	.564	22	.209	.421
16	.204	.012	04	.188	.482	24	.105	.849	01	.326	.037	29	.264	.284	22	.232	.953	16	.221	.311
08	.208	.418	02	.208	.577	07	.139	.159	30	.334	.938	11	.282	.262	14	.259	.217	29	.235	.356
19	.211	.798	03	.214	.402	01	.175	.641	22	.405	.295	14	.379	.994	01	.275	.195	28	.264	.941
29	.233	.070	07	.245	.080	23	.196	.873	05	.421	.282	13	.394	.405	06	.277	.475	11	.287	.199
07	.260	.073	15	.248	.831	26	.240	.981	13	.451	.212	06	.410	.157	02	.296	.497	02	.336	.992
17	.262	.308	29	.261	.087	14	.255	.374	02	.461	.023	15	.438	.700	26	.311	.144	15	.393	.488
25	.271	.180	30	.302	.883	06	.310	.043	06	.487	.539	22	.453	.635	05	.351	.141	19	.437	.655
06	.302	.672	21	.318	.088	11	.316	.653	08	.497	.396	21	.472	.824	17	.370	.811	24	.466	.773
01	.409	.406	11	.376	.936	13	.324	.585	25	.503	.893	05	.488	.118	09	.388	.484	14	.531	.014
13	.507	.693	14	.430	.814	12	.351	.275	15	.594	.603	01	.525	.222	04	.410	.073	09	.562	.678
02	.575	.654	27	.438	.676	20	.371	.535	27	.620	.894	12	.561	.980	25	.471	.530	06	.601	.675
18	.591	.318	08	.467	.205	08	.409	.495	21	.629	.841	08	.652	.508	13	.486	.779	10	.612	.859
20	.610	.821	09	.474	.138	16	.445	.740	17	.691	.583	18	.668	.271	15	.515	.867	26	.673	.112
12	.631	.597	10	.492	.474	03	.494	.929	09	.708	.689	30	.736	.634	23	.567	.798	23	.738	.770
27	.651	.281	13	.499	.892	27	.543	.387	07	.709	.012	02	.763	.253	11	.618	.502	21	.753	.614
04	.661	.953	19	.511	.520	17	.625	.171	11	.714	.049	23	.804	.140	28	.636	.148	30	.758	.851
22	.692	.089	23	.591	.770	02	.699	.073	23	.720	.695	25	.828	.425	27	.650	.741	27	.765	.563
05	.779	.346	20	.604	.730	19	.702	.934	03	.748	.413	10	.843	.627	16	.711	.508	07	.780	.534
09	.787	.173	24	.654	.330	22	.816	.802	20	.781	.603	16	.858	.849	19	.778	.812	04	.818	.187
10	.818	.837	12	.728	.523	04	.838	.166	26	.830	.384	04	.903	.327	07	.804	.675	17	.837	.353
14	.895	.631	16	.753	.344	15	.904	.116	04	.843	.002	09	.912	.382	08	.806	.952	05	.854	.818
26	.912	.376	01	.806	.134	28	.969	.742	12	.884	.582	27	.935	.162	18	.841	.414	01	.867	.133
28	.920	.163	22	.878	.884	09	.974	.046	29	.926	.700	20	.970	.582	12	.918	.114	08	.915	.538
03	.945	.140	25	.939	.162	05	.977	.494	16	.951	.601	19	.975	.327	03	.992	.399	25	.975	.584

Col. No. 8			Col. No. 9			Col. No. 10			Col. No. 11			Col. No. 12			Col. No. 13			Col. No. 14		
A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C
09	.042	.071	14	.061	.935	26	.038	.023	27	.074	.779	16	.073	.987	03	.033	.091	26	.035	.175
17	.141	.411	02	.065	.097	30	.066	.371	06	.084	.396	23	.078	.056	07	.047	.391	17	.089	.363
02	.143	.221	03	.094	.228	27	.073	.876	24	.098	.524	17	.096	.076	28	.064	.113	10	.149	.681
05	.162	.899	16	.122	.945	09	.095	.568	10	.133	.919	04	.153	.163	12	.066	.360	28	.238	.075
03	.285	.016	18	.158	.430	05	.180	.741	15	.187	.079	10	.254	.834	26	.076	.552	13	.244	.767
28	.291	.034	25	.193	.469	12	.200	.851	17	.227	.767	06	.284	.628	30	.087	.101	24	.262	.366
08	.369	.557	24	.224	.572	13	.259	.327	20	.236	.571	12	.305	.616	02	.127	.187	08	.264	.651
01	.436	.386	10	.225	.223	21	.264	.681	01	.245	.988	25	.319	.901	06	.144	.068	18	.285	.311
20	.450	.289	09	.233	.838	17	.283	.645	04	.317	.291	01	.320	.212	25	.202	.674	02	.340	.131
18	.455	.789	20	.290	.120	23	.363	.063	29	.350	.911	08	.416	.372	01	.247	.025	29	.353	.478
23	.488	.715	01	.297	.242	20	.364	.366	26	.380	.104	13	.432	.556	23	.253	.323	06	.359	.270
14	.496	.276	11	.337	.760	16	.395	.363	28	.425	.864	02	.489	.827	24	.320	.651	20	.387	.248
15	.503	.342	19	.389	.064	02	.423	.540	22	.487	.526	29	.503	.787	10	.328	.365	14	.392	.694
04	.515	.693	13	.411	.474	08	.432	.736	05	.552	.511	15	.518	.717	27	.338	.412	03	.408	.077
16	.532	.112	20	.447	.893	10	.476	.468	14	.564	.357	28	.524	.998	13	.356	.991	27	.440	.280
22	.557	.357	22	.478	.321	03	.508	.774	11	.572	.306	03	.542	.352	16	.401	.792	22	.461	.830
11	.559	.620	29	.481	.993	01	.601	.417	21	.594	.197	19	.585	.462	17	.423	.117	16	.527	.003
12	.650	.216	27	.562	.403	22	.687	.917	09	.607	.524	05	.695	.111	21	.481	.838	30	.531	.486
21	.672	.320	04	.566	.179	29	.697	.862	19	.650	.572	07	.733	.838	08	.560	.401	25	.678	.360
13	.709	.273	08	.603	.758	11	.701	.605	18	.664	.101	11	.744	.948	19	.564	.190	21	.725	.014
07	.745	.687	15	.632	.927	07	.728	.498	25	.674	.428	18	.793	.748	05	.571	.054	05	.797	.595
30	.780	.285	06	.707	.107	14	.745	.679	02	.697	.674	27	.802	.967	18	.587	.584	15	.801	.927
19	.845	.097	28	.737	.161	24	.819	.444	03	.767	.928	21	.826	.487	15	.604	.145	12	.836	.294
26	.846	.366	17	.846	.130	15	.840	.823	16	.809	.529	24	.835	.832	11	.641	.298	04	.854	.982
29	.861	.307	07	.874	.491	25	.863	.568	30	.838	.294	26	.855	.142	22	.672	.156	11	.884	.928
25	.906	.874	05	.880	.828	06	.878	.215	13	.845	.470	14	.861	.462	20	.674	.887	19	.886	.832
24	.919	.809	23	.931	.659	18	.930	.601	08	.855	.524	20	.874	.625	14	.752	.881	07	.929	.932
10	.952	.555	26	.960	.365	04	.954	.827	07	.867	.718	30	.929	.056	09	.774	.560	09	.932	.206
06	.961	.504	21	.978	.194	28	.963	.004	12	.881	.722	09	.935	.582	29	.921	.752	01	.970	.692
27	.969	.811	12	.982	.183	19	.988	.020	23	.937	.872	22	.947	.797	04	.959	.099	23	.973	.082

Table B-1 Random Numbers for General Sampling Procedure (Continued)

Col. No. 15			Col. No. 16			Col. No. 17			Col. No. 18			Col. No. 19			Col. No. 20			Col. No. 21		
A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C
15	.023	.979	19	.062	.588	13	.045	.004	25	.027	.290	12	.052	.075	20	.030	.881	01	.010	.94
11	.118	.465	25	.080	.218	18	.086	.878	06	.057	.571	30	.075	.493	12	.034	.291	10	.014	.93
07	.134	.172	09	.131	.295	26	.126	.990	26	.059	.026	28	.120	.341	22	.043	.893	09	.032	.34
01	.139	.230	18	.136	.381	12	.128	.661	07	.105	.176	27	.145	.689	28	.143	.073	06	.093	.18
16	.145	.122	05	.147	.864	30	.146	.337	18	.107	.358	02	.209	.957	03	.150	.937	15	.151	.01
20	.165	.520	12	.158	.365	05	.169	.470	22	.128	.827	26	.272	.818	04	.154	.867	16	.185	.45
06	.185	.481	28	.214	.184	21	.244	.433	23	.156	.440	22	.299	.317	19	.158	.359	07	.227	.27
09	.211	.316	14	.215	.757	23	.270	.849	15	.171	.157	18	.306	.475	29	.304	.615	02	.304	.40
14	.248	.348	13	.224	.846	25	.274	.407	08	.220	.097	20	.311	.653	06	.369	.633	30	.316	.07
25	.249	.890	15	.227	.809	10	.290	.925	20	.252	.066	15	.348	.156	18	.390	.536	18	.328	.79
13	.252	.577	11	.280	.898	01	.323	.490	04	.268	.576	16	.381	.710	17	.403	.392	20	.352	.28
30	.273	.088	01	.331	.925	24	.352	.291	14	.275	.302	01	.411	.607	23	.404	.182	26	.371	.21
18	.277	.689	10	.399	.992	15	.361	.155	11	.297	.589	13	.417	.715	01	.415	.457	19	.448	.75
22	.372	.958	30	.417	.787	29	.374	.882	01	.358	.305	21	.472	.484	07	.437	.696	13	.487	.59
10	.461	.075	08	.439	.921	08	.432	.139	09	.412	.089	04	.478	.885	24	.446	.546	12	.546	.64
28	.519	.536	20	.472	.484	04	.467	.266	16	.429	.834	25	.479	.080	26	.485	.768	24	.550	.03
17	.520	.090	24	.498	.712	22	.508	.880	10	.491	.203	11	.566	.104	15	.511	.313	03	.604	.78
03	.523	.519	04	.516	.396	27	.632	.191	28	.542	.306	10	.576	.659	10	.517	.290	22	.621	.93
26	.573	.502	03	.548	.688	16	.661	.836	12	.563	.091	29	.665	.397	30	.556	.853	21	.629	.15
19	.634	.206	23	.597	.508	19	.675	.629	02	.593	.321	19	.739	.298	25	.561	.837	11	.634	.90
24	.635	.810	21	.681	.114	14	.680	.890	30	.692	.198	14	.749	.759	09	.574	.599	05	.696	.45
21	.679	.841	02	.739	.298	28	.714	.508	19	.705	.445	08	.756	.919	13	.613	.762	23	.710	.07
27	.712	.366	29	.792	.038	06	.719	.441	24	.709	.717	07	.798	.183	11	.698	.783	29	.726	.58
05	.780	.497	22	.829	.324	09	.735	.040	13	.820	.739	23	.834	.647	14	.715	.179	17	.749	.91
23	.861	.106	17	.834	.647	17	.741	.906	06	.848	.866	06	.837	.978	16	.770	.128	04	.802	.18
12	.865	.377	16	.909	.608	11	.747	.205	27	.867	.633	03	.849	.964	08	.815	.385	14	.835	.31
29	.882	.635	06	.914	.420	20	.850	.047	03	.883	.333	24	.851	.109	05	.872	.490	08	.870	.54
08	.902	.020	27	.958	.856	02	.859	.356	17	.900	.443	05	.859	.935	21	.885	.999	28	.871	.53
04	.951	.482	26	.981	.976	07	.870	.612	21	.914	.483	17	.863	.220	02	.958	.177	25	.971	.36
02	.977	.172	07	.983	.624	03	.916	.463	29	.950	.753	09	.863	.147	27	.961	.980	27	.984	.25

Col. No. 22			Col. No. 23			Col. No. 24			Col. No. 25			Col. No. 26			Col. No. 27			Col. No. 28		
A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	
12	.051	.032	26	.051	.187	08	.015	.521	02	.039	.005	16	.026	.102	21	.050	.952	29	.042	.0
11	.068	.980	03	.053	.256	16	.068	.994	16	.061	.599	01	.033	.886	17	.085	.403	07	.105	.2
17	.089	.309	29	.100	.159	11	.118	.400	26	.068	.054	04	.088	.686	10	.141	.624	25	.115	.4
01	.091	.371	13	.102	.465	21	.124	.565	11	.073	.812	22	.090	.602	05	.154	.157	09	.126	.6
10	.100	.709	24	.110	.316	18	.153	.158	07	.123	.649	13	.114	.614	06	.164	.841	10	.205	.1
30	.121	.744	18	.114	.300	17	.190	.159	05	.126	.658	20	.136	.576	07	.197	.013	03	.210	.0
02	.166	.056	11	.123	.208	26	.192	.676	14	.161	.189	05	.138	.228	16	.215	.363	23	.234	.5
23	.179	.529	09	.138	.182	01	.237	.030	18	.166	.040	10	.216	.565	08	.222	.520	13	.266	.7
21	.187	.051	06	.194	.115	12	.283	.077	28	.248	.171	02	.233	.610	13	.269	.477	20	.305	.6
22	.205	.543	22	.234	.480	03	.286	.318	06	.255	.117	07	.278	.357	02	.288	.012	05	.372	.2
28	.230	.688	20	.274	.107	10	.317	.734	15	.261	.928	30	.405	.273	25	.333	.633	26	.385	.1
19	.243	.001	21	.331	.292	05	.337	.844	10	.301	.811	06	.421	.807	28	.348	.710	30	.422	.3
27	.267	.990	08	.346	.085	25	.441	.336	24	.363	.025	12	.426	.583	20	.362	.961	17	.453	.7
15	.283	.440	27	.382	.979	27	.469	.786	22	.378	.792	08	.471	.708	14	.511	.989	02	.460	.9
16	.352	.089	07	.387	.865	24	.473	.237	27	.379	.959	18	.473	.738	26	.540	.903	27	.461	.8
03	.377	.648	28	.411	.776	20	.475	.761	19	.420	.557	19	.510	.207	27	.587	.643	14	.483	.0
06	.397	.769	16	.444	.999	06	.557	.001	21	.467	.943	03	.512	.329	12	.603	.745	12	.507	.3
09	.409	.428	04	.515	.993	07	.610	.238	17	.494	.225	15	.640	.329	29	.619	.895	28	.509	.7
14	.465	.406	17	.518	.827	09	.617	.041	09	.620	.081	09	.665	.354	23	.623	.333	21	.583	.8
13	.499	.651	05	.539	.620	13	.641	.648	30	.623	.106	14	.680	.884	22	.624	.076	22	.587	.9
04	.539	.972	02	.623	.271	22	.664	.291	03	.625	.777	26	.703	.622	18	.670	.904	16	.689	.3
18	.560	.747	30	.637	.374	04	.668	.856	08	.651	.790	29	.739	.394	11	.711	.253	06	.727	.2
26	.575	.892	14	.714	.364	19	.717	.232	12	.715	.599	25	.759	.386	01	.790	.392	04	.731	.8
29	.756	.712	15	.730	.107	02	.776	.504	23	.782	.093	24	.803	.602	04	.813	.611	08	.807	.9
20	.760	.920	19	.771	.552	29	.777	.548	20	.810	.371	27	.842	.491	19	.843	.732	15	.833	.7
05	.847	.925	23	.780	.662	14	.823	.223	01	.841	.726	21	.870	.435	03	.844	.511	19	.896	.4
25	.872	.891	10	.924	.888	23	.848	.264	29	.862	.009	28	.906	.367	30	.858	.299	18	.916	.3
24	.874	.135	12	.929	.204	30	.892	.817	25	.891	.873	23	.948	.367	09	.929	.199	01	.948	.6
08	.911	.215	01	.937	.714	28	.943	.190	04	.917	.264	11	.956	.142	24	.931	.263	11	.976	.7
07	.946	.065	25	.974	.398	15	.975	.962	13	.958	.990	17	.993	.989	15	.939	.947	24	.978	.6